# 9th International Conference on Durability of Materials and Components

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# Implementation Of The European Construction Products Directive Via The Iso 15686 Standards

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**Summary: In 1993, on a significant European initiative, a working group aimed at developing standards for design life was established in ISO/TC59, Building Construction. This was identified as a guiding concept regarding durability of building products that should be of help in implementing the European Construction Products Directive, CPD. The establishment was based on the Vienna Agreement on co-operation between CEN and ISO. Subsequently, the working group was elevated to ISO/TC59/SC14.**

**The scope of the activity is to document steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life without incurring large unexpected expenditures. The activity has progressed to the stage where the first two parts of the developing standard, ISO 15686 Buildings and Constructed Assets – Service Life Planning, are approved, i.e. Part 1, General principles and Part 2, Service life prediction procedures. Part 3, Performance audits and reviews, is soon expected to be approved. Additional parts are being drafted or have been proposed.**

**In Europe, the members of the European Organisation for Technical Approvals, EOTA, are now starting to issue European Technical Approvals for products as one of the two possible routes for CE marking them. This is done on the basis of an assessment of the product that includes the evaluation of its durability in order to obtain a reasonable economical working life, as required by the CPD. The durability evaluation is performed according to general EOTA guidance developed on the basis of the service life prediction concepts as expressed in ISO 15686-2.**

**For product standards – the other possible route for CE marking – this durability evaluation process is not yet so advanced, although attempts of implementation are starting in the concrete area. However, a similar development for other groups of materials in general is not equally straightforward. To facilitate for writers of product standards to implement service life issues in accordance with ISO 15686, a general guidance document is badly needed.**

**The development of ISO product standards addressing service life issues will not only be beneficial on the European level for the implementation of the CPD, but also on the world-wide level to obtain useful and recognised tools for communication of building products' service life characteristics.**

**Keywords: Construction Products, Service Life, Technical Approvals, Standards.**

## 1 INTRODUCTION

In 1993, the standardisation work in the field of service life planning started when ISO/TC59/SC14/WG9, Design Life of Buildings, was launched at a meeting in Atlanta. The purpose of the activity is to document steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life, the design life, without incurring large unexpected expenditures. It is needed to facilitate the making of objective estimates of the service lives of buildings and facilities.

There was a significant European initiative to establishing the standardisation group. The EUREKA umbrella project Eurocare has as its strategic goal to increase the service life of the built environment and to decrease yearly life-cycle costs for its conservation, restoration and maintenance. Eurocare, therefore, early adopted the Service Life Methodology established by CIB and RILEM (Masters and Brandt 1989). Together the three organisations in 1991 took the initiative to a dialogue with the Commission of the European Communities and the European standardisation body CEN regarding standardisation of the Service Life Methodology.

This was identified as a guiding concept regarding durability of building products that should be of help in implementing the European Construction Products Directive, CPD (The Council of the European Communities 1988), (Caluwaerts et al. 1993). ISO/TC59/SC14/WG9 was the result of a CEN/BTS-1 request and based on the Vienna Agreement on co-operation between CEN and ISO. Subsequently, as the scope and work broadened, in 1998 the working group was elevated to the subcommittee level, obtaining the designation ISO/TC59/SC14.

In Europe, the members of the European Organisation for Technical Approvals, EOTA, are now starting to issue European Technical Approvals for products as one of the two possible routes for CE marking them. This is done on the basis of an assessment of the product that includes the evaluation of its durability in order to obtain a reasonable economical working life, as required by the CPD. The durability evaluation is performed according to general EOTA guidance (EOTA 1999a) developed on the basis of the service life prediction concepts as expressed in ISO 15686-2.

For product standards, the other possible route for CE marking, this durability evaluation process is not yet so advanced, although attempts of implementation are starting in the concrete area. However, a similar development for other groups of materials in general is not equally straightforward. To facilitate for writers of product standards to implement service life issues in accordance with ISO 15686, a general guidance document is badly needed.

Of course, the development of ISO product standards addressing service life issues will not only be beneficial on the European level for the implementation of the CPD, but also on the world-wide level to obtain useful and recognised tools for communication of building products' service life characteristics.

## 2 STATE OF WORK OF ISO/TC59/SC14, DESIGN LIFE

The activity has progressed to the stage where the first two parts of the developing standard, ISO 15686 Buildings and Constructed Assets – Service Life Planning, are approved, i.e. Part 1, General principles (ISO 2000a) and Part 2, Service life prediction procedures (ISO 2001). Part 3, Performance audits and reviews (ISO 2000b) is soon expected to be approved.

Part 1 describes the principles and procedures that apply to design, when planning the service life of buildings and constructed assets. It is important that the design stage includes systematic consideration of local conditions to ensure, with a high degree of probability, that the service life will be no less than the design life. The standard is applicable to both new constructions and the refurbishment of existing structures. However, additional considerations may apply to existing buildings.

Part 2 of the standard series is mainly based on the Service Life Methodology (Masters and Brandt 1989). It describes a procedure that facilitates service life predictions of building components. The general framework, principles, and requirements for conducting and reporting such studies are given. It should be emphasised that the standard does not describe the techniques of service life prediction of building components in detail.

Part 3 has reached the DIS status and is intended to become a fully approved standard in June 2000. This part is concerned with ensuring the effective implementation of service life planning. It describes the approach and procedures to be applied to pre-briefing, briefing, design, construction and, where required, the life care management and disposal of buildings and constructed assets to provide a reasonable assurance that measures necessary to achieve a satisfactory performance over time will be implemented. However, the cost implications of service life planning and the broader issues of sustainability (e.g. embodied energy, land use) are not addressed here.

Other parts on which work has commenced are on "Data requirements", "Life cycle costing" and "Guidelines for considering environmental impacts". The first of these is intended to describe requirements of data in order to estimate the service life of a structure, building system, or building. However, at least in a first step, this part will be developed as a technical report rather than a standard. The second work item, "Life-cycle costing", is to enable comparative assessment of the cost performance of buildings and constructed assets over an agreed period of time. The last work item is to provide guidance on assessing the relative environmental impacts of alternate service life designs, and to identify the interface between environmental LCA and service life planning.

Another recently approved work item is "Maintenance and condition assessment protocols for buildings", with the objective to develop guidance for improving the quality of durability and service life data derived from condition assessment of the existing building stock. Still another work item, "Reference service life", aiming to provide guidance on the provision of reference service life for use in the application of ISO 15686-1, is currently a subject of balloting.

Moreover, proposals for new work items on maintenance and on characterisation of degradation environments are likely to be drafted.

Finally, a new work item proposal on service life design of concrete structures is under way. Originally initiated via discussions at a workshop arranged amongst others by CEN/TC104, Concrete, this work item was brought to consideration by SC14 and is now formally transferred to ISO/TC71, Concrete, Reinforced Concrete and Pre-Stressed Concrete. Thus, the recently established ISO/TC71/SC7, Service Life Design of Concrete Structures has, in liaison with *fib*, started to prepare the work item proposal with the aim of developing a Code of practice on service life design of concrete structures. This would become the first international product standard addressing service life planning issues explicitly, simultaneously serving as an example or a template how to cope with other building products in this context.

Figure 1 gives an overview of the ISO 15686 series of standards produced and in process. The M1 and P1 levels in the figure depict the generic standards on service life planning: Service life prediction procedures and general principles, respectively. The levels P2 and M2 represent semi-generic support standards while the M3 level illustrates the manifold of product standards that in time should be complemented with descriptions of service life assessment procedures. The proposed work item on service life design of concrete structures is intended to serve as an example of such a product standard.

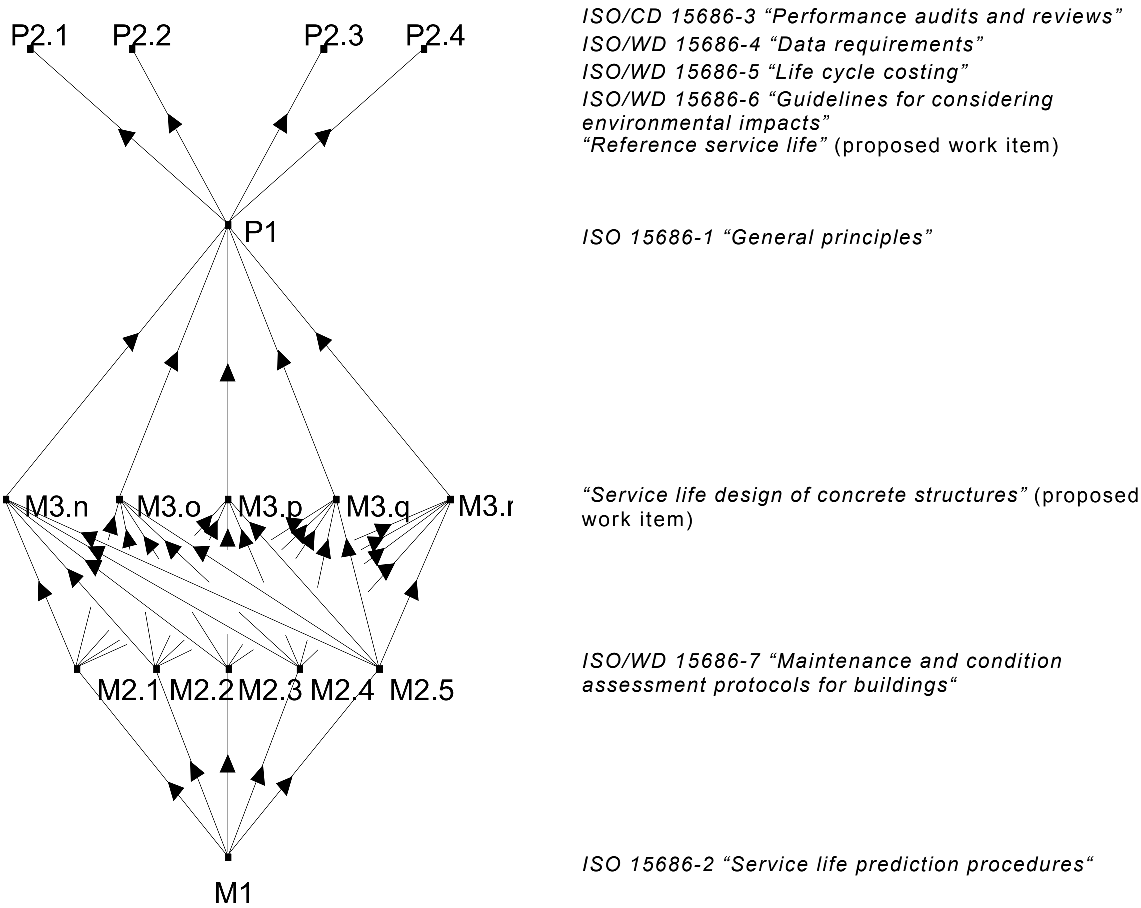


**Figure 1. Relationship among standards for service life design and planning of buildings and constructed assets – ISO 15686 series.**

The major work and challenge in this standardisation area lies in completing existing product standards with sub-parts describing methods for assessment and declaration of service life data. Another vital work area is methods for exposure/testing and evaluation. The major parts of this work do not lie within the scope of SC14, but they ought to meet the requirements of, and be developed in accordance with the framework of the SC14 standards.

## 3 THE EUROPEAN CONSTRUCTION PRODUCTS (CPD) DIRECTIVE

In 1988 the EU Council adopted the Construction Products Directive (The Council of the European Communities 1988) with the aim of removing the barriers to trade in this specific area following the white paper on the internal market approved by the Council in June 1985. The CPD is different from other similar so-called “New Approach” directives, such as the ones on toys or machines, in as far as it tries to accommodate with the specificity and complexity of the construction sector.

Indeed the CPD fixes the essential requirements for the construction works (both buildings and civil engineering works). There are six essential requirements which apply to the works as set out in annex I of the directive:

- Mechanical resistance and stability;
- Safety in case of fire;
- Hygiene, health and the environment;

- Safety in use;
- Protection against noise;
- Energy economy and heat retention, and;
- Such requirements must, subject to normal maintenance, be satisfied for an economically reasonable working life.

The CPD has, however, not the aim of harmonising the regulations on works of the Member States but rather of bridging the "works" and the construction products put on the European market and used in these works. It therefore specifies the level of performances of these products: the products "have such characteristics that the works in which they are to be incorporated, assembled, applied or installed, can, if properly designed and built, satisfy the essential requirements...". This is the notion of "fitness for the intended use" of a construction product.

The link between the essential requirements of the works and the product characteristics to be assessed is fixed in Interpretative Documents (The European Commission 1994). The Interpretative Documents (IDs) have been drafted on the basis of existing regulatory requirements of the Member States, each ID containing necessarily one chapter on the requirements of the works and one on the characteristics of the products, in principle allowing to accommodate the requirements for the works.

Since the essential requirements of the works may present differences according to climate, geographical location or regulations from a country or a region, so may also product characteristics, if needed, be expressed in classes and levels.

The CPD specifies further that a construction product is fitted for its intended use if it conforms to:

- a harmonised European standard (drafted by CEN/CENELEC);
- a European Technical Approval (issued by an EOTA member), or;
- a non-harmonised technical specification (e.g. a national technical specification) recognised at Community level,
- all three being denoted technical specifications.

Finally the CPD specifies that, before getting a CE marking, the conformity to these technical specifications must be attested. This will be carried out via certification by a third party or self-declared by the manufacturer. It involves anyhow factory production control procedures to be applied by the manufacturer. Details are described in the systems of attestation of conformity (AC) as laid down in Annex III of the CPD. For each family of products, the AC system is fixed in a Commission Decision, after consultation of the Standing Committee on Construction (representatives of the Member States). With respect to the assumption of the working life of construction products in harmonised technical specifications, reference is made to the following provisions of the CPD and the IDs, fixing the working context both for CEN and EOTA:

- According to the CPD "the products must be suitable for construction works which (as a whole and in their separate parts) are fit for their intended use, account being taken of economy, and in this connection satisfy the following essential requirements where the works are subject to regulations containing such requirements. Such requirements must, subject to normal maintenance, be satisfied for an economically reasonable working life".
- *Maintenance* is defined in the IDs as "a set of preventive and other measures that are applied to the works in order to enable the works to fulfil all its functions during its working life".
- *Normal maintenance* according to the IDs "includes inspections and occurs at a time when the costs of the intervention which has to be made are not disproportionate to the value of the part of the works concerned, consequential costs being taken into account".
- The *Working life* is defined in the IDs as "the period of time during which the performance of the works will be maintained at a level compatible with the fulfilment of the Essential Requirements".
- *Economically reasonable working life* according to the IDs "presumes that all relevant aspects are taken into account such as: costs of design, construction and use; costs arising from hindrance of use; risks and consequences of failure of the works during its working life and costs of insurance covering these risks; planned partial renewal; costs of inspections, maintenance, care and repair; costs of operation and administration; disposal; environmental aspects".
- The IDs provide that it is up to the Member States "when and where they feel it necessary, to take measures concerning the working life which can be reasonable for each type of works, or for some of them, or for parts of the works, in relation to the satisfaction of the essential requirements".
- Furthermore, the IDs provide that category B specifications (product standards) and Guidelines for European Technical Approval (ETAGs) "should include indications concerning the working life of the products in relation to the intended uses and the methods for its assessment".

## 4 EUROPEAN TECHNICAL APPROVALS

As described above harmonised European (product) standards and European Technical Approvals are both technical specifications in the sense specified by the CPD. For most products, performance will be described via product standards supported by test standards or horizontal standards. The CPD, therefore, recognises this route via standardisation as the main route for the European harmonisation and to CE marking for construction products. Harmonised European (product) standards are developed by CEN/CENELEC under mandate of the EC and EFTA. The CPD, however, recognises also that for other products this route is not applicable:

For all those products where the state of the art does not or not yet permit to produce a product standard, the CPD has envisaged an alternative route to allow also these products to come to the market and to overcome their barriers to trade: this route is the ETA. Products for which an ETA can be granted are specified in Article 8 of the CPD.

An ETA is a favourable assessment of the fitness of a product for an intended use, based on the fulfil-ment of the essential requirements for building works for which the product is used. An ETA is placed at the same level as a harmonised European standard: it must be followed by an attestation of confor-mity under a given system before the CE marking can be affixed to the product. An ETA for a product of a manufacturer can be granted on the basis of an ETA Guideline for a product family drafted by EOTA after having received a mandate from the Commission and the EFTA Secretariat (Article 11 of the CPD) or on the basis of an internal EOTA procedure for "isolated" products (Article 9.2).

Having to fulfil the principles of the CPD in the drafting of ETAGs and in the issuing of ETAs, EOTA has worked out a number of guidance rules for the handling of the above mentioned concepts on working life of construction products in ETAGs and related ETAs *and for the assessment of their durability*.

Firstly and based on the work of ISO, a table (refer to Table 1) has been drafted indicating the working life of construction products to be assumed in ETAGs and ETAs depending on assumed working lives of the works (e.g. given by national regulations). It should be noted that, according to the IDs, the working life of a product "cannot be interpreted as a guarantee given by the producer, but regarded only as a means for choosing the right products in relation to the expected economically reasonable working life of the works". By EOTA the assumed working life of a product should be understood as a basic assumption and reference to be considered when laying down the type and severeness of verification methods (e.g. number of freeze-thaw cycles) and provisions relating to durability *in the ETAGs and related ETAs.* When allocating given products to working life categories, the presumptions of the IDs concerning the "economically reasonable working life" should be taken into account.

**Table 1. Relation between assumed working life of construction works and of products**

| Assumed working life of works (years) | | Working life of construction products to be assumed in ETAGs, ETAs and hENs (years) | | |
|---|---|---|---|---|
| Category | Years | Repairable or easily replaceable[1] | Repairable or replaceable with some more efforts | Lifelong[2] |
| Short | 10 | 10 | 10 | 10 |
| Medium | 25 | 10 | 25 | 25 |
| Normal | 50 | 10 | 25 | 50 |
| Long | 100 | 10 | 25 | 100 |

[1] In exceptional and justified cases, e.g. for certain repair products, a working life of 3 to 6 years may be envisaged (when agreed by EOTA TB or CEN respectively).

[2] When not repairable or replaceable "easily" or "with some more efforts".

Secondly general guidance has been produced to ETAG writers on the approach that should be taken in the development of *these ETAGs* on the subject of Assessment and/or Prediction of Working Life of products, in order to achieve a consistent and harmonised technical approach between different *ETAGs*, and to limit the amount of long term ageing to be performed during assessments *of products leading to their CE marking*.

The approach taken is based on the methodology of (ISO 2001), see Figure 2, and briefly described in clause 8 of (ISO 2001), but adapted to the specificity of technical approval work, in particular for the *quick* assessment of innovative products:

1) In the *Definition* stage, use is made of assumptions as to the "normal" worst conditions that the product will see in use. These conditions are not usually the absolute worse conditions but are chosen so that the majority of the product population will be at or below these conditions. As many individual products will be at conditions less severe than the "normal" it is obvious that many products will achieve working lives longer than that predicted, in some instances by factors of 2 or 3.

   The term "User needs" relates, essentially, to the definition of what is expected of the product and relates to the product's fitness for use. In the context of the work of EOTA this will be defined, within limits set by the EOTA Working Group and its Mandate, by the applicant for an ETA by his claims made for the product. Assessments should be limited to user needs only in so far they relate to the Essential Requirements of the CPD.

The "Building Context" relates to aspects such as the climate and/or site in which the building will be located, the effects of its occupancy and use, incorporation into the works etc. Where the precise building context is not known, or where more than one context is anticipated by the applicant, a "worst case" situation should be assumed. Alternatively, each context should be considered separately. Further guidance is given on possible building context sub-divisions:

- climatic sub-divisions of Europe
- other external sub-divisions such as orientation of the product in structures and positions of the building
- internal environment sub-division
- other internal sub-divisions (orientation and use of the building or compartments of buildings)
- subterranean

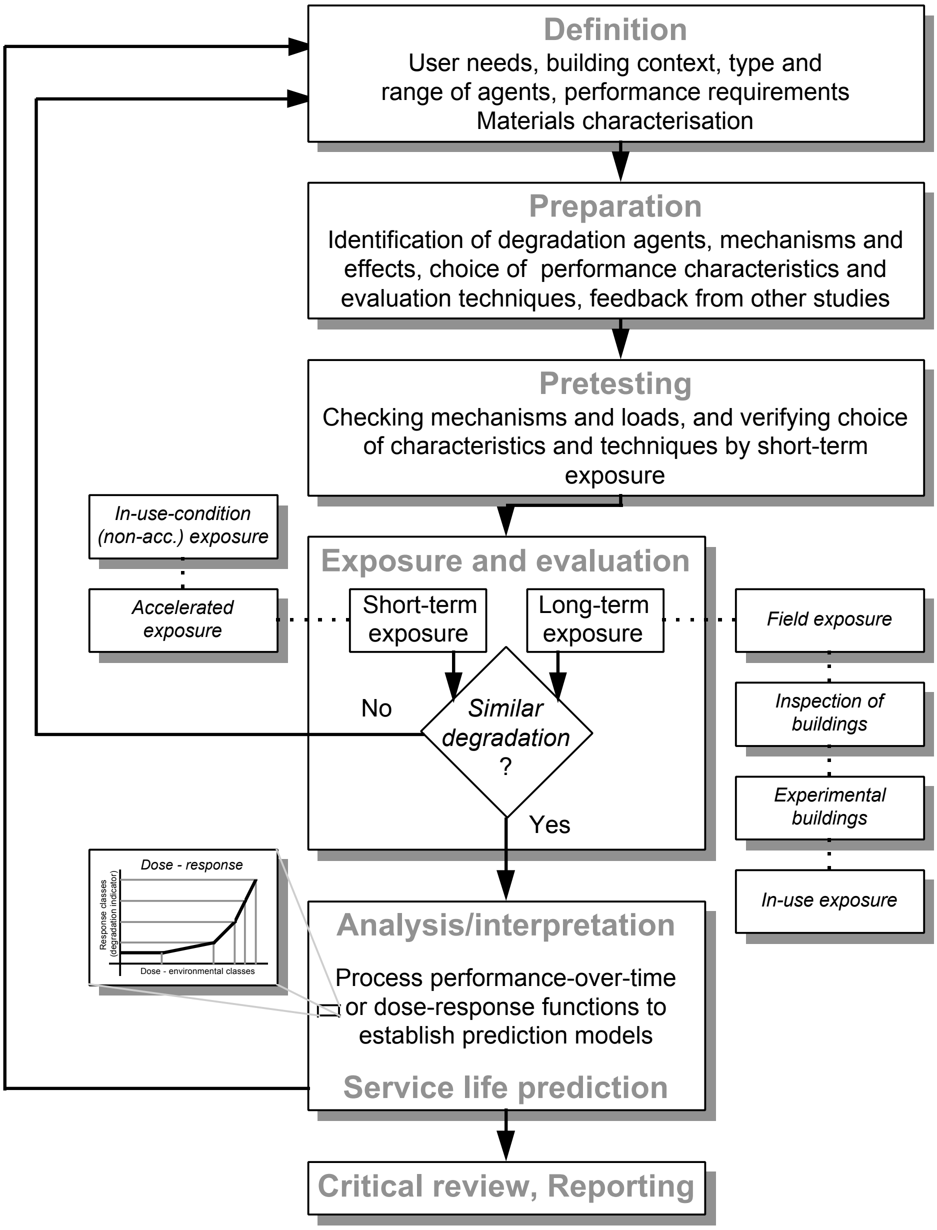


**Figure 2. Systematic methodology for service life predictions of building materials and components (ISO 2001)**

Performance requirements for particular products or components are defined by the Essential Requirements of the CPD, expanded upon by the Interpretative Documents and laid down in the EC mandates, based upon the regulatory requirements applicable in the Member States. The appropriate performance requirements, performance criteria and methods of verification to be used in the assessment of working life are elaborated upon in the ETA Guideline.

Finally and in order to ensure that a predicted working life relates to the product on the market it is essential that all products are adequately characterised in terms of structure, chemical composition and performance values corresponding to the selected performance criteria.

2) The *Preparation* stage is the most critical stage when drafting the ETAG. It involves the identification of the possible degradation factors (e.g. weathering, biological, incompatibility, use etc), identification of the degradation mechanisms and the effects of degradation on the product. The result will be the proposal or selection of the most appropriate approach (including ageing tests).

Therefore, possible degradation factors and their effects in relation to building context are listed (see examples in Tables 2 and 3). This list is not meant to be exhaustive and relates mainly to the effects of degradation factors on materials. Also other factors, such as use-related factors, the possible effects of interaction between the product and other parts of the building (e.g. due to dimensional changes) as well as the possibility of synergistic effects between two or more factors which may cause a greater than predicted change (e.g. UV radiation + moisture) should be taken into account.

**Table 2. Example of degradation factors that have to be taken into account: (exposure situation internal- 1st part)**

| Degradation factor | Actions | Reactions | Materials possibly at risk (examples) | Sub-division of factor |
|---|---|---|---|---|
| **Solar radiation U.V.** | *Products are generally protected from UV radiation* | | | |
| **Thermal** | Solar heat gain causing rise in internal temperature[1,2] | **Thermal expansion**<br>- bowing or twisting<br>) temporary<br>) permanent<br>- loss of bond<br>**Cyclic expansion/contraction**<br>- fatigue damage | Metals<br>Plastics<br>Thin sheet materials<br>Multilayer materials<br>Concrete<br>Mechanically fixed product<br>Bonded products | Climatic zones<br>Orientation of building or building compartment |
| **Differential temperatures**<br>Internal/external[3] | Thermal expansion/contraction | **Mechanical**<br>- bowing<br>- twisting<br>- delamination | Thin panels<br>Multilayer materials | Climatic zones<br>Internal conditions |
| Internal/internal[4] | Thermal expansion/contraction | **Mechanical**<br>- bowing<br>- twisting<br>- delamination | Thin panels<br>Multilayer materials | Internal conditions |
| **Localised heating[5]** | Localised chemical/physical degradation<br>Localised thermal expansion contraction | **Chemical**<br>- embrittlement<br>- change in appearance<br>**Physical**<br>- embrittlement<br>- change in appearance<br>- loss of thermal properties<br>**Mechanical**<br>- bowing<br>- distortion<br>- delamination | Plastics<br>Thin panels<br>Multilayer materials | Temperature of heat source<br>Continuous/intermittent |
| **Depressed temperatures[6]** | Thermal contraction | **Mechanical**<br>- shrinkage | Materials generally | |

[1] Only significant for unventilated buildings or building compartments

[2] Most significant in uninsulated lightweight structures

[3] Refrigerated buildings – special case

[4] Separation of heated/unheated rooms

[5] Materials behind radiators etc or special cases such as factories with elevated process temperatures

[6] As for external in refrigerated buildings and unheated buildings or parts of buildings – minimum temperatures may be different

**Table 3. Risks related to degradation factor and orientation (internal)**

| Orientation of internal degr. factor | Horizontal surfaces | Vertical surfaces | Ceilings |
|---|---|---|---|
| Water | High risk of liquid water | Low risk of liquid water | Low risk of liquid water |
| | Risk of condensation | Risk of condensation | Risk of condensation |
| Chemical spillage | High risk | Low risk | Low risk |
| Wear | High risk | Low risk | No risk |
| | - pedestrians | different form of wear | |
| | | - people brushing against | |
| Impacts | High risk - dropping impacts | Horizontal impacts from people, vehicles etc | Small risk, special cases e.g. sports arenas |

For developing a programme for the assessment of working life, technical specification writers will have to make use of many sources of information used to identify and select the most appropriate assessment procedure taking into account the nature of the product and its intended use: knowledge and experience (the chemistry and physical make up of the material and a knowledge of the proposed use), natural exposure data of construction products either under service conditions or under defined exposure conditions, testing following accelerated ageing (falling into three main groups: direct, indirect and torture tests) taking into account specific accelerated ageing conditions and extrapolation rules.

3) In the methodology envisaged by (ISO 2001), *Pretesting* is designed to validate the proposed ageing test. In the context of ETA work (unique and quick assessment of a product of a manufacturer wishing to bring his product as soon as possible to the market) this stage will generally not be possible. It is therefore essential that full use is made of all existing knowledge and data held by Approval Bodies and others when drafting the ETAGs and having to evaluate the products.
4) As far as the testing step is concerned, ETAGs describe fully any ageing test proposed, including conditions, periods, performance criteria etc. Tests should be as short term as possible and should generally be related to a fixed period of exposure, with the overriding requirement that it allows the prediction of working life with an acceptable degree of confidence.

The ISO methodology anticipates the comparison of the results of relatively short-term tests to those of long term tests under in-service conditions and a dose effect loop. This will not be possible in the case of assessments for ETAs and consequently ETAG writers should, wherever possible, utilise existing and generally accepted tests. However, the review procedures envisaged for ETAGs and ETAs, and the associated increase in knowledge and experience of a product, will give them the opportunity to re-assess the methods of verification selected for the determination of working life.

## 5 CONCLUSIONS/FUTURE WORK

The international standardisation on the design life of buildings essentially sprung out from a European initiative to support the implementation of the Construction Products Directive. Similar motives and needs were apparent in most parts of the world, encompassing performance based building and construction issues, the need for standards and tools for the service life planning and design of buildings, and in general environmental and resource conservation concerns.

The major parts of the future standardisation work lie in amending the various product standards with guides on the assessment and evaluation of durability and service life of products when in normal use in constructed assets. This is a long term perspective and work. The European standardisation organisation CEN has not reached too far in this process. CEN's prime and immediate objective for durability assessment so far is to use the present state-of-the-art in each product family (The European Commission 2001). This results in standards addressing durability and working lives of products at very unequal levels: some are or will be very detailed, whilst others will give nothing or only basic points without details.

In future generations of harmonised product standards, a more systematic approach following the ideas developed in this paper ought to be introduced. ISO/TC59/SC14 is discussing to process a generic standard giving guidance on how service life assessment and declaration of service life data ought to be addressed in product standards to meet the requirements of the already established standards, and hence the Construction Products Directive. Furthermore, the initiative of TC59/SC14 to start a standardisation action in the field of service life design of concrete structures, should be seen as a first example of an implementation, setting the level of ambition for others.

A promising new approach that might be appointed in future product standards for complementing the classical approach of durability studies was the presented at the 8DBMC Conference, and subsequently improved (Lair et al. 2001). This approach suggests two steps in the process of service life prediction. One is aimed of taking benefit from all information available, from experts' judgements to durability test results. In order to anticipate premature failure or degradation, the predicted service life thus obtained may then be corrected by an FMEA (failure mode and effects analysis) exercise.

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# Durability In The Built Environment And Sustainability In Developing Countries

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**Summary: This paper discusses the relevance of sustainable construction and the role of durability and service life on sustainability. It summarizes the concept of service life planning, in accordance with ISO 15686, and investigates the applicability of this concept to the construction industry in developing countries. Examples and statistical data are given. Conclusions are presented.**

**Durability and service life have very important environmental dimensions. In some languages sustainable construction is translated as durable construction. Durability is much more knowledge-based than resource-based, because it is possible to improve service life without increasing environmental loads during the production phase.**

**Service life planning implies using of estimated service life of components as additional design criteria. The concept offers a logical scheme to make rational decisions based on economical (life cycle costs) and environmental (life cycle assessment) aspects.**

**More than 82% of the world's population lives in so-called developing countries. These countries still need to construct a great number of houses and infrastructure. In other words, the developing world is still under construction and, potentially, can benefit very much from service life planning tools. However, several constraints like low environmental concern, the self-help construction scheme, higher interest rates and a lack of technical data and experts constitute barriers.**

**Keywords: Sustainability, service life planning, developing countries, durability**

## 1 SUSTAINABILITY AND CONSTRUCTION

Sustainability has many definitions, but most people agree that it implies producing goods with a lower environmental load, thereby helping to preserve the environment for future generations. The Rio Summit 1992 definition includes additional dimensions to the concept, aside from the purely environmental one. First, the social dimension, which implies a more equal distribution of the results of development, within a country and between all countries, and second, the democratic dimension that demands increasing the direct participation of the public in the decision making process (John, 2001). All these dimensions are equal in importance.

### 1.1 Sustainability and construction

The construbusiness is the biggest raw material consumer of any society. Kasai (1998) estimates that construction consumes about 50% of the total material consumption in Japan. Matos & Wagner (1999) estimate that in the USA construction is responsible for 75% of the total consumption of materials. The transformation of these raw materials into goods and the frequently long transportation distances require an extensive amount of additional resources and results in significant environmental loads. Additional resources are consumed during usage of the built facilities, plus those required by maintenance and decommissioning activities.

As a result, the sector is also responsible for consuming a significant part of the energy water and for the generation of pollution (John, Agopyan & Sjöström, 2001). For example, the decomposition of limestone during the production of the clinker Portland is solely responsible for generating about 3% of the world's $CO_2$ generation.

Construction and demolition are also major sources of waste. Estimates of construction and demolition wastes vary worldwide from a mere 163 kg per capita to 3 658 kg per capita, with typical values surpassing 400kg per capita, which are typical values for domestic solid waste generation (John, 2000). To this total it is necessary to ad all residues generated during building materials production.

Buildings are also important consumers of energy. WRI (2000) estimates that the residential, commercial and public sectors are responsible for 34,5% of the world's total energy consumption.

### 1.2 Increasing sustainability of the industry

Sustainability certainly implies a *delinking* development from environmental loads (Janseen; Van Der Bergh, 1999). One specific tool for *delinking* is *dematerialization.* This implies delivering the same performance with smaller amounts of materials, which in turn means a reduction in pollution generation, transportation and decommissioning, and a reduction of the consumption of raw materials.

Dematerialization can also be achieved by replacing the linear model of production, where natural raw materials are extracted from nature, industrially transformed into products which are used and, after the end of service life are discharged as waste along with the residues of the production processe. The new production paradigm is often called *closed loop* (Curwell & Cooper, 1998) or *cyclical production* (Craven *et all*, 1996). Within the building sector this implies that the production of buildings be maintained and operated with a minimum of resources, easily upgraded and/or refurbished. When the building becomes obsolete it will be disassembled and the resulting building components reused in a "new" building or, if they have already reached the end of their service life, recycled (John, 2000,). The amount of raw materials consumed and waste generated would be greatly minimized. Maximizing the use of renewable energy and renewable materials are, perhaps, the most commonly known other tools for reducing environmental load.. Moving from carbon-based energy to a clean and renewable energy source will clearly solve global warming and acid rain problems. But the only source for renewable building materials is vegetable: wood, lignocellulosic fibres, etc. It is possible to expand the use of vegetable materials in construction, but obviously they cannot replace all other building materials, at least not without major industrial transformations. Additionally, even all the cultivable land available on Earth would not be enough to supply wood for the construction industry (Curwell; Cooper, 1998).

Recycling waste as raw material is also an important tool for preserving natural resources, and very frequently, reduces energy consumption (John & Zordan, 2001). Making the new products easily recyclable after the end of their service life is also very important.

Increasing durability is also an option for disconnecting materials consumption from development and, therefore, reducing environmental loads.

## 2 DURABILITY AND SUSTAINABILITY

Durability, measured as the in use service life distribution of a population of components, plays a very important role in achieving sustainable construction. Bordeau (1999), when reporting different concepts of sustainable construction collected by the CIB W82 among different countries, observes that in some languages like Dutch, Finnish, Romanian or French "sustainable" is translated as "durable".

This confusion between sustainability and durability makes sense, because "one way of extending resource productivity is by extending the useful life of products" (DeSimone & Poppof, 1998). Increasing the service life can also reduce the amount of construction and demolition waste. Equation (a) describes the relationship between the environmental dimension of sustainability and service life:

$$S = \frac{\sum El}{ESL} \tag{a}$$

where $S$ is the environmental sustainability index; $El$ is the sum of all environmental loads of the building, on a cradle-to-grave basis; and $ESL$ the (estimated) service life of the building. When comparing two solutions, which have an equal environmental load, e.g. they use the same amount of a specific material; the more sustainable is the one that has the longer service life. So, solutions with short service lives can be environmentally sustainable.

Of course, the proposed equation does not take into consideration the social and democratic dimensions of sustainability. In the long run, shorter service lives imply using more resources in maintenance, decommissioning and in reconstruction instead of using those "limited resources" to enlarge the existing built environment. Even if the short service life solution is the more environmentally sustainable, it may not be socially sustainable because it hampers enlargement of the built environment, e.g. housing, roads and sanitation. In developing countries enlargement of the built environment is a condition for achieving social sustainability and also for increasing national productivity.

### 2.1 Durability, a knowledge based attribute

Durability is much more knowledge-based than resource-based. Very frequently most of the environmental load is originated by the production of the component and service life can be increased or reduced without affecting the environmental loads proportionally. Some examples and evidence support this concept.

Let's consider a reinforced concrete structure in an urban environment. The most common degradation mechanism will be corrosion of the steel bars due to carbonation of the concrete (CEB, 1993). Carbonation depth is a function of the square root of time and mechanical strength and a small increase of concrete cover from 20 to 25 mm can result in an increase of the service life from 50 years to 78 years, an increase of 56%. Applying that to a concrete component with a cross-section of 25 x 25 cm, this increase of 56% on the service life will cause an increase of only 8,2% on the environmental load during construction. Sometimes an increase in the service life of reinforced concrete structures can be achieved while reducing the environmental loads. Durability on marine environments can be improved by replacing clinker Portland with residues like granulated blast furnace slag or fly ash (Bijen, 1999). The durability of plastic components frequently depends on very small changes in its formulation (Guillermo Martinez, 1996).

Durability is not an intrinsic quality of a material. Simple changes in design details that result in greater protection of component from degradation factors can increase its service life with little or no change of the environmental load.

Obsolescence is another durability problem (ISO, 1999). Obsolescence is not the result of a degradation process but from changes in the user needs. It can be seen as socially defined service life. Since it is impossible to forecast social changes in the long run, "durability" against obsolescence cannot be controlled. But the environmental loads associated with obsolescence can be minimized, making components more likely to be obsolete easy replaceable. Additionally, the entire building will eventually become obsolete. Design decisions can control the speed of the obsolescence of the building and make the disassembling process and the reuse of the components easy. Knowledge is the key factor to control obsolescence related durability problems.

## 3 SERVICE LIFE PLANNING ACCORDING TO ISO 15686

ISO 15686-1 defines service life planning as a "preparation of the brief and design for the building and its parts to achieve the desired design life e.g., in order to reduce the costs of building ownership and facilitate maintenance and refurbishment." (Annex B – Alphabetical glossary of terms). Its introduction starts by saying " Service life planning is a design process which seeks to ensure … that the service life of a building will equal or exceed its design life, while taking into account (and preferably optimising) the life cycle costs of the building".

Assuring that the building will reach the designed service life, defined as the one "intended by the designer e.g., as stated by the designer to the client to support specification decisions" is an essential part of service life planning. The second part of the service life planning process is to optimise the life cycle costs. Life cycle costs (LCC) are defined as the total cost at present value of a building or its parts throughout its life, including the costs of planning, design, acquisition, operations, maintenance and disposal, less any residual value. It deals with the economic performance of a building.

Even though LCC concept is a major incentive to adopt service life planning practices, the environmental dimension of service life planning is introduced only on item 10 (Financial and environmental costs over time).

Service life planning introduces the forecasting of service life into the design phase. It includes the following steps: (a) definition of the design service life of the building; (b) forecasting the estimated service life of building components; (c) using the estimated service life as a decision criteria for selecting alternatives during the design, in order to minimize life cycle costs.

The forecasting of the estimated service life of the component (ESLC) is carried out multiplying the reference service life of the component (RSLC) by modifying factors (A to G).

$$ESLC = RSLC \times A \times B \times C \times D \times E \times G$$

The modifying factors are those which to take into account the effect of the differences between the forecasted in-use conditions and those conditions observed during the evaluation of the RSLC. The considered differences are relative to (A) quality of components; (B) design level; (C) work execution level; (D) indoor environment; (E) outdoor environment; (F) in-use conditions; (G) maintenance level.

Ideally, the component's manufacturer will provide the reference service life data of the component. Inside the European Union the availability of this data is enforced by the EU Construction Products Directive 89/106/EEC (The Council of the European Communities. 1998). Other sources can also be used to do service life forecasting, such as previous experience or observation of similar constructions or materials in similar conditions, assessments of durability in Technical Approval certificates, results published in books, building codes, etc. A generic methodology to assess the reference service life is also provided.

Service life planning is intended to make the building as durable as that desired by the owner. It will allow maintenance activities to be planned during the design phase, which makes room for design decisions that makes these activities easy to perform. Also it should make easy refurbishment activities, defined as modification and to an existing building or its parts to bring it up to an acceptable condition. Service life planning makes it possible to integrate LCC and LCA during the design stage.

Two conditions are essential for applying the service life planning concept. Knowledge and infrastructure for measuring the reference service life, as well as knowledge of the effect of each factor on the reference service life.

# 4 SERVICE LIFE PLANNING, SUSTAINABILITY AND CONSTRUCTION IN DEVELOPING COUNTRIES

## 4.1 Sustainability and the construction industry in developing countries

Sustainability is a global concept for answer global problems, but its application in societies or regions results in very diverse Agendas 21. CIB's recent effort on developing a version of its Agenda 21 for Sustainable Construction adapted to developing countries is recognition of this fact. More than 82% of the world's population lives in developing countries (World Bank, 2001).

Developing countries are under construction because they, lack an adequately built environment. In the 43 countries with highest income 90% of the roads are already paved. In Latin America and the Caribbean only 24.2% of the roads are paved (World Bank, 2000). The lack of adequate houses is another indicator. Brazil alone needs to build about 5 million houses to be able to offer suitable shelter for its own citizens. In other words, expansion of the construction activities aiming to create an adequately built environment for all citizens is one of the items on the national Agenda 21.

The impact of the *construbusiness* on the environment in third world countries is relatively more important than it is in the developed countries. For example, the liberation of $CO_2$ due to decomposition of calcium carbonated during the production of clinker Portland is responsible for more than 6% of the total $CO_2$ released in Brazil but only accounts for about 1% in developed countries like the USA and Canada (Fig 1).

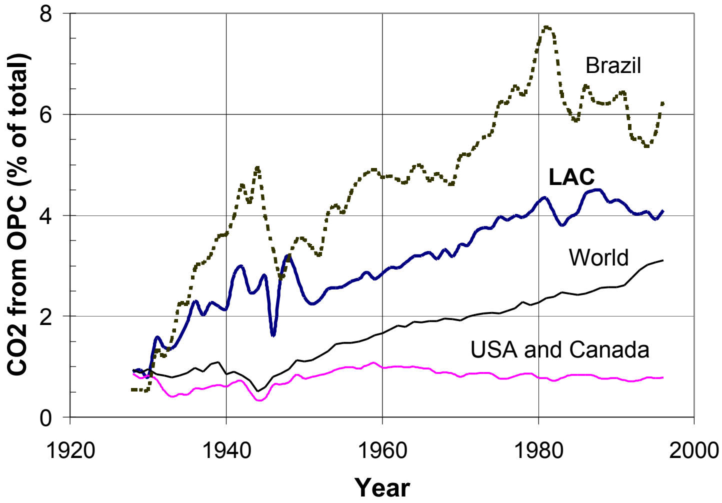


**Figure 1. Participation of clinker decomposition of limestone during clinker Portland production in the total $CO_2$ released in Latin America and the Caribbean, Brazil, Worldwide, and the USA and Canada (original data from Marland, Boden & Andres, 2000). These figures do not include $CO_2$ due to combustion.**

Statistical data about construction and demolition waste generation is scarce in developing countries. In Brazil typical generation rates are around 486 kg/year per person, considerably higher than domestic solid waste generation (John, Agopyan & Sjöström, 2001). In Chile the estimates are 236 kg/year per person.

It seems accurate to conclude that sustainable construction is even more important in developing countries than in the developed parts of the world.

## 4.2 Constraints for service life planning and sustainable actions

### 4.2.1 Lower environmental concern

Large portions of the population in developing countries, are hard pressed by basic problems such as hunger, poverty, violence, financial problems and scarcity of infrastructure. As a result, the developing societies tend to be less concerned about the environment than those from developed countries (Ocampo, 1999). Due to a scarcity of public resources to improve the infrastructure, different groups compete to build more facilities as quickly as possible, without much concern about quality and sustainability.

Even the very basic tools for protecting the environment are not yet implemented. In all of Latin America and the Caribbean, only Brazil, Chile and Mexico are considered to already have an acceptable measuring network for the monitoring of urban air quality (Kork; Saénz, 1999). In addition, there is a lack of basic legislation for waste management and most of the existing landfills do not comply with sanitary standards (Acurio *et al.*, 1997).

Ocampo (1999) pointed out a lack of effective authority among the environmental public agencies as an additional barrier to implementing sustainable policies in Latin America.

As a result, the Agenda 21 in Developing Countries tends to be very much focused on poverty, democracy, increasing infrastructure and preservation of natural resources.

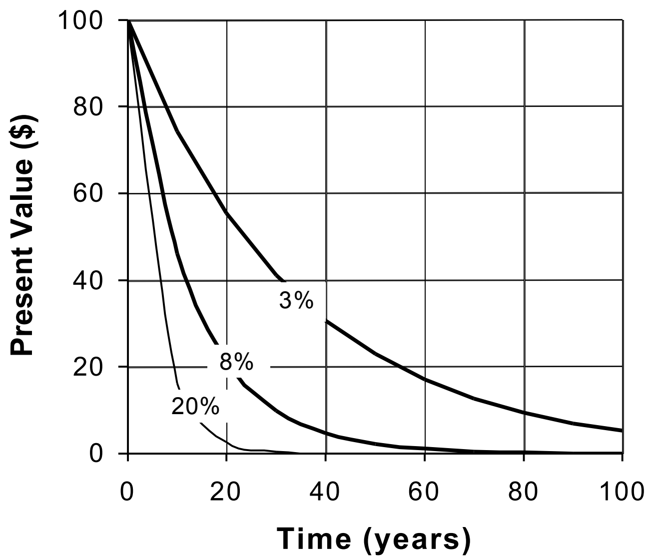


**Figure 2. Effect of annual discount rate on the present value of $100 expense made in future**

### 4.2.2 Scarcity of financial resources

Governments and societies of developing countries do not have enough money to meet the demands of better built-environments. This applies to governments as well to individual consumers. Also, interest rates in developing world tend to be much higher than in the developing world. Interest rates for building and construction in Brazil have historically been above 10% a year and the government has been paying interest rates of around 20% a year for last few years. For life cycle costs based decisions (as well as all other decisions made from a financial point of view) the higher is the discount rate the less important future expenses will be (Fig 2), because the financial decisions are made considering present values[1] (ASTM, 1999). Therefore they tend to emphasize the cost of construction and neglect long-term costs and even construction facility service life. Hence, service life planning tends to be discouraged due to financial reasons.

### 4.2.3 The self-help housing

In most developing countries the informal construction sector is very important. In Latin America and the Caribbean (LAC) a very significant portion of the houses are built by the family members themselves. In Bolivia, this kind of production scheme is responsible for 45 to 55% of the total urban houses produced every year (CLICHEVSKY, 2000). In Brazil, most of the building materials like masonry blocks, ceramic tiles, Portland cement and paint are sold directly to individual consumers and not to building contractors. This reality has profound consequences on the building materials market. Price is the major driving force because most consumers do not have the technical knowledge to demand quality, not to mention to judge the environmental impact of different alternatives.

The self-help houses are in most cases no not designed in the traditional sense, since no engineers or architects are engaged. There is no of human resources training, nor quality control. Without technical support it is not possible to adopt sustainable practices, including service life planning, which requires specialized knowledge.

### 4.2.4 Continuous upgrading

Low cost houses produced by the government are as a rule small and very simple. However, most of the houses are upgraded by the owners (John et al. 1995, Du Plessis, 1999). This process starts as soon as the owner is able to divert some money to home improvement and continuous for the life of the house. Some low-cost housing projects clearly exploit this concept by producing so called building embryos, i.e. a very small house. This building embryo normally consists of a small multi-

[1] It is important to note that the environmental impact of an activity is not reduced because it is planned for future occasions. This is the essential difference between life cycle costs (LCC) and life cycle assessment (LCA) methodologies. Therefore, as emphasized by ISO/DIS 15686-1, LCC are not directly related to LCA.

functional room and a toilet and is supposed to be enlarged by the owner. The same upgrading process occurs with the houses produced by self-help schemes, whose construction almost never ends.

The typical upgrading process includes improving aesthetics of the buildings, either by changing the façade as described by Du Plessis (1999) or by applying on new and more sophisticated interior finishing. Enlarging the house is an essential characteristic of this process. New rooms or an extra bathroom, a garage, a second or even third floor are part of this process, which can also include improvements on building services. Eventually this continuous upgrading of each house results in improvement of the entire neighborhood making it attractive to wealthier people. These new and wealthier users then take the upgrading process a step further.

ISO 15686-1 does not define the term upgrading but uses it together with refurbishing: "Whether allowance has been made for replacement, maintenance and/or upgrading to avoid undue disruption to the use of the building." item 6.2 Conceptual and initial design); and "Refurbishment and upgrading are the major strategies to counter obsolescence" (item 11.3 – Minimizing obsolescence). In both situations, upgrading and refurbishment are tools to prevent obsolescence. But the two terms are considered to be different in some way.

Upgrading as described here is results from an improvement in the economical situation of the owners. It is implicit in the definition that the user was not satisfied with the previous performance level of the house but was forced to accept it due to financial constraints. In this context the ISO 15686-1 definition of obsolescence "loss of ability of an item to perform satisfactorily due to changes in performance requirements" seems to not fully apply. Also, ISO 15686-1's description of economic obsolescence is related to high running or maintenance costs, which is very different from the previously described upgrading situation.

### 4.2.5 Lack of technical data and experts

Developing countries do not invest in research and development to the same extent that developed countries do. The research community in LAC, for example, is small and the national investment in research below 0.8% of the GNP (Hill, 2000).

Finally, developing countries focus less research on durability of building materials and components. Less than 12% of the papers presented at the 8DBMC, Vancouver 1999, originated from developing countries, including Turkey, Kuwait, India, China, Brazil, Israel, Botswana and Eastern European countries. Consequently, within developing countries little is known about the durability of building materials or service life planning. Even in technical communities, knowledge about degradation mechanisms and service life forecasting is not widespread. In general, building materials manufacturers in developing countries lack incentives to deliver reference service life data.

## 5 CONCLUSIONS

Durability and service life have very important environmental dimensions. Achieving longer service lives is more dependent on knowledge than on resources.

Service life planning and service life forecasting, as proposed by ISO 15686 are important tools for integrating the durability concept during the design stage, thereby allowing for life cycle costs estimates and life cycle assessment.

Since developing countries are under construction, they can, potentially, benefit more from the concept of Service Life Planning than the developed ones. However some important constraints make the adoption of this concept difficult.

A very important part of the construction activity in developing countries is done through self-help housing, with no technical assistance. As a rule, there is a continuous process of upgrading low-cost housing.

A scarcity of financial resources places the emphasis on construction costs and not on long-term costs. High interest rates in developing countries make solutions with low initial costs and higher maintenance and running costs more economically attractive. Even solutions with very short service lives can be financially attractive.

Developing countries lack data on durability and the local building materials manufacturers have few incentives to deliver it.

## 6 ACKNOWLEDGEMT

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# Specifications For Road Construction And Maintenance With Warranty Contracts

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**Summary**: **Several road agencies around the world are exploring the use of warranty type contracts in road construction and maintenance. International practices show that the specifications used in warranty type contracts are significantly different to the conventional approaches. In this paper, some key issues in developing specifications for warranty contracting are identified. The issues addressed include criteria for product selection, duration of the warranty, performance indicators and thresholds, quality control/quality assurance, remedial actions, conflict resolution team, warranty bond and payment, etc**

**Keywords. Warranty, specifications, road construction, maintenance**

## 1 INTRODUCTION

Budgetary constraints and increased public awareness of road conditions have encouraged road agencies to examine alternative road construction contracting methods. In the last decade, various innovative contracting processes, such as design-build, A+B Biding, lane rental, warranty and job order contracting have been implemented in road construction industry (UTTC 2000). International practices indicate that warranty contracts can reduce on-going maintenance costs, encourage contractors' innovation, and eventually benefit road users (Kumar 2001, Russell *et al* 1999). This paper is intended to address some key issues in developing specifications for warranty road construction and maintenance.

Warranty is a guarantee of the integrity of a product and of the makers' responsibility for the replacement or repair of deficiencies. Historically, warranties are widely used in manufactured products. European road agencies have used various types of warranty road contracts for almost two decades. Table 1 gives a summary of current warranty practices in several European countries.

**Table 1 European warranty practices**

| *Country* | *Structural design* | *QA/QC* | *Warranty period* | *Warranty terms* |
|---|---|---|---|---|
| Austria | State approved | Contractor | 2-5 years | Warranty bond |
| Denmark | State | Contractor | 5 years min | 5% retention |
| France | Contractor | Contractor | 10 years | Failure paid by contractor |
| Germany | Contractor (within state established limits) | QA-State<br>QC-Contractor | 4-5 years | 5% retention |
| Norway | State | Contractor | 3 years | 15% warranty bond |
| Sweden | Joint | Contractor | 3-5 years | Failure paid by contractor |

(Source Stephens *et al* 1998)

In North America, the first modern warranty contract for road construction was used in the North Carolina Department of Transportation for a pavement marking project in 1987. Since then, more and more state highway agencies in North America have started to adopt this approach. Until 1997, over 240 warranty projects have been completed in the United States (Russell *et al* 1999). In Australia and New Zealand, several state road agencies have started to adopt the process on a trial basis in recent years.

## 2 TYPES OF SPECIFICATIONS IN ROAD CONSTRUCTION AND MAINTENANCE

### 2.1 Method and material-based specifications

Traditionally, method and material-based specifications have been used by highway agencies to control construction and maintenance quality. In theory, these specifications reduce job delays and contract claims, and contract compliance is easily determined. However, there are two major disadvantages. Firstly, it gives no opportunity and motivation for contractors to improve the construction processes or the final constructed product. Secondly, full-time presence of experienced personnel from contracting agencies is required for proper enforcement.

### 2.2 Quality control/quality assurance specifications

Quality control/quality assurance was developed to primarily improve the method and material-based processes. As used in industrial manufacturing, the specifications, based on representative sampling and statistical acceptance procedure, outline contractors' strategies and procedures for controlling quality. Contractors are allowed to use preferred means and method to provide the end result to meet the statistical acceptance requirements of the owners. To some extent, it can take advantages of the contractors' experience and innovation. The constraint of quality control/quality assurance specifications is that the specifications are absolute acceptance and rejection. In reality, there is a gray area in which the product is below the acceptance criteria but still provides value to the owners. This largely depends upon the type of product.

### 2.3 Performance–related specifications

Performance–related specifications were developed to provide an improvement over quality control/quality assurance specifications. Performance–related specifications describe the desired levels of fundamental engineering properties that are predictors of performance and appear in primary performance relationships. The specifications intend to quantify and link the properties of the materials and construction process to the performance of the end product. However, researchers (Ohrn & Schexnayder 1998) have acknowledged that the primary relationships involved in developing performance–related specifications were more complicated than anticipated. More work is required in establishing performance prediction models that can realistically quantify the effects of the variation of material properties and construction quality on the performance of the end product.

### 2.4 Warranty specifications

Under an ideal warranty contract, the specification is expressed directly in terms of desired performance parameters and thresholds at various times throughout the warranty period. The contractors are given the freedom of materials selection and construction methods. Under current practices, warranty specifications are generally written as a combination of quality control/quality assurance specification and performance-related specification (Russell *et al* 2000). The details are addressed in the following sections.

### 2.5 End result specifications

Transportation Research Board (TRB) described the end result specifications as "specifications that required the contractor to take the entire responsibility for supplying a product or an item of construction. The highway agency's responsibility is to either accept or reject the final product or apply a price adjustment that compensates for the degree of compliance with the specifications" (Benson 1995). Generally, the specifications can be found in Design-Build-Operation, Design-Build-Operation-Transfer contracting approaches.

The distinctive difference between end result specification and performance–related specification is that the former focuses on the measurements of the end product and the latter measures the indicators of material properties and quality during the construction process. Under an end result specification, the contractor takes fully responsibility in deciding what to do and how.

## 3 POTENTIAL BENEFITS AND CONSTRAINTS OF USING "WARRANTY" IN ROAD CONSTRUCTION AND MAINTENANCE

### 3.1 Potential benefits

When using warranty specifications, the final product is assessed in terms of performance parameters directly related to the road users. Therefore, contractors are expected to provide a safe and usable roadway for public rather than to simply meet prescriptive standards on construction materials and methods.

In comparing with traditional contracting processes, warranty contracting relocates some post-construction performance risks between the contractor and contracting agency. Therefore, a higher accountability for construction quality is passed on to the contractor. Contractors are in a better position to manage the day-to-day quality due to their direct relationship with suppliers and subcontractors and their direct control over construction activities. They have more interest in adopting a new construction method if it offers possible competitive advantage. Theoretically, a higher quality is more likely to be achieved by using warranty contracting.

Under this process, contractors have to a large extent, the freedom to select materials and construction methods. Therefore the agency is no longer simply purchasing the contractor's labor, equipment, and material, but is also ultilising contractor's experience and innovation. The process stimulates contractors to improve their construction techniques in order to maximise profits. Any cost savings arisen from such innovation will also be passed on to the agency and the community.

International practices shows that warranty contracting decreases the demands on owner's human and physical resources (Russell *et al* 2000, Stephens *et al* 1998). Due to contractors taking responsibility of quality control/quality assurance, there are less demands on daily in-field inspection during construction period. In post-construction stage, the situation is also improved because contractor has to repair and correct possible deficiencies during the warranty period.

### 3.2 Problems and constraints

Current warranty specifications are the combination of quality control/quality assurance specification and performance–related specifications. Although extensive work has been done to predict performance behavior of pavements, the relationships are still being tested and verified.

Theoretically, the length of warranty period is generally determined by the "time to identify the deficiencies of the end products". However, in some cases, the determination of the duration of the contract is based on the discussion between the contracting community and the owner. The warranty periods vary from 2 to 20 years. As agencies gain more experience, the duration may be changed.

Experiences in other industry sectors, such as the pipeline industry, indicate that even though warranty specifications are detailed, installation in some cases may not occur to these specifications. Therefore, the construction supervisors may still play an important role in quality assurance in critical works such as gas industry.

Compared to traditional approaches, warranty processes prolong the contracting duration that does increase contracting administration tasks for both agencies and contractors. Most agencies ask for bonds or retain parts of the bid amount for the warranty works. Small contractors may be eliminated from the market due to financial incapability for warranty projects. Even for large companies, ongoing increased outstanding warranty obligations may result in difficulties to get bond for new projects.

In general, warranty approach is new for the road construction industry. With the increase of warranty type contracts, its long-term impact on cost savings and performance of end product will be better identified.

## 4 KEY ISSUES IN DEVELOPING WARRANTY SPECIFICATIONS FOR ROADWORKS

Previous research (Russell *et al* 1999) has proposed a framework with eleven key elements for warranty specifications. However, some important issues in developing a warranty specification should be further investigated.

### 4.1 Product selection criteria

In current practices, the warranty products for road construction and maintenance include asphalt pavement, chip seal, micro-surfacing, concrete pavement, bridge components, bridge painting, pavement marking, etc. Generally, the product for warranty should have the following attributes (UTTC 2000):

a) Performance of the product is measurable and quantifiable. The failure thresholds can be well defined in the specification.

b) The warranty work is wholly controlled by the contractor, and allows the contractor to select optimal design, construction process and material.

c) The factors beyond contractor's control have minimal impact on the warranty product, or the possible defects associated can be well identified.

### 4.2 Duration of warranty

The warranty periods vary from 2 to 20 years. It is generally accepted that five years is an acceptable evolution period to assure the performance of an asphalt product, without overburdening the contractors. Typical warranty periods for chip seal and micro surfacing are about 3 and 2 years, respectively. In some cases, the warranty period is determined by dialogue between agency and contracting community. However, care should be taken whether the agreed period is appropriate for deficiency identification. A comprehensive reviewing of past maintenance records is essential before finalising the agreement.

### 4.3 Warranty bond and payment

Most agencies ask for bond or retain some percentage of contract amount for remedial work guarantee. In Indiana, USA, a bond is required. The amount is based on the cost to remove and replace the warranty asphalt work. Michigan, USA, keeps

10% of the contract lump sum price of the contract item as the bond. Similar to Michigan, partial contract price is withheld in Ohio, however, the percentage is varied based on the course thickness (as given in Table 2).

**Table 2 Percent of retainment versus course thickness in Ohio**

| *Course thickness* | Percentage |
|---|---|
| 2.0 inches (50mm) or less | 90% |
| 2.1 to 4.0 inches (51 to 100mm) | 60% |
| 4.1 inches (101mm) or more | 30% |

(Source ODOT 1999 b)

**Table 3 Michigan warranty payments versus year**

| Period Year | Payment as % of Contract LSUM |
|---|---|
| 1 | 0 |
| 2 | 1 |
| 3 | 2 |
| 4 | 3 |
| 5 | 4 |

(Source MDOT 1995)

According to a survey conducted by Stephen *et al.* (1998), the bond issue is one of the major concerns in contracting community. Small and medium size contractors believed that they would face financial difficulties with accumulative bond requirements. The bond agents also expressed concern on their long-term financial conditions from prolonged project duration. It suggests that an ongoing dialogue on bond issue between highway agencies and contracting community will be critical for successful implementation of warranty approach.

Warranty payment can be made annually. Normally, the payment is made based upon an annual inspection of the pavement made by the agency to determine its condition relative to the warranty performance criteria. In Michigan, the performance payments are made within 30 days after the anniversary date of the acceptance of the pavement construction. The percentage of payment for each year is given in Table 3.

### 4.4 Conflict resolution team

Study undertaken by Russell *et al* (1999) found that half of the state agencies in North America had a conflict resolution team in warranty projects. The scope of the team includes all issues related to material selection, quality control plan, distress rate, and remedial works. This team is especially useful for partial reconstruction, chip seal (spray seal), and micro-surfacing projects. In these projects, the contractor has to accept pre-existing conditions that may result in the damage of the warranty products.

Indiana (USA) conflict resolution team consists of two contractor representatives, two department representatives, and a fifth person mutually agreed upon by both parties. In case of dispute, the team will give a final recommendation by a majority vote in which each member has an equal vote. The cost for the fifth member will be equally shared between the agency and the contractor.

### 4.5 Performance indicators and thresholds

The selection of performance parameters is important in developing a warranty specification (Kumar 2001). For road users, ride quality and safety are two major concerns. The agency, in addition, expects that the products contribute to low life-cycle cost. Performance indicators should include the above expectations. The performance indicators generally fall into three categories: safety, ride quality, and quality performance.

Skid resistance is generally accepted as an appropriate safety measure for pavement running surface. Standard methods such as the friction number test, have been used world wide.

One widely used measure of ride quality is Present Serviceability Index (PSI). The drawback of this approach is that it is based on subjective opinion of the evaluation panel. With the promotion of the World Bank models, there is a growing interest in the highway industry for using International Roughness Index (IRI). This objective measure can be acquired by standard equipment at highway travel speed.

Quality performance indicators represent the structural adequacy and durability of the warranty product. They also provide information from a maintenance perspective for the agency. The suitable parameters are rutting, cracking, surface defects, etc. The performance indicators currently used in some state agencies in North America are given in Table 4.

Thresholds of performance indicators can be determined by reviewing the historical records of agency's pavement management system. Generally, the condition of pavement will decline with accumulative traffic load. For a certain volume of traffic, it is appropriate to define different performance levels at various times throughout the warranty period. For example, Michigan gives the maximum number of distress points for any segment during each year of warranty period (as given in Table 5).

The thresholds are determined based on specific traffic conditions. In the specification, attention should be given to account for possible change of traffic condition. Indiana releases contractor's warranty obligation when the measured accumulative number of Equivalent Single Axle Loads (ESAL's) exceeds the design value by 20 percent or more.

### 4.6 Warranty works

4.6.1 Pre-existing conditions

The contractor has to accept some pre-existing conditions that can result in possible pavement failure. If the specifications do not define pre-existing conditions that could lead to premature failure beyond contractor's control, it is expected that disputes will arise. In Ohio (ODOT 1999a), the warranty does not apply to structural problems below the chip seal (spray seal), the structural problem is not the fault of the contractor.

**Table 4 Current warranty practices in North America**

| *State* | *Warranty product* | *Warranty period* | *Performance indicators* | *Threshold levels* | *Warranty Bond* |
|---|---|---|---|---|---|
| Indiana (USA) | Asphalt | 5 years | Roughness(IRI) | 2.1 m/km IRI | Bond: Lump sum amount |
| | | | Friction Number | 25 | |
| | | | Rutting | 9 mm | |
| | | | Longitudinal Cracking | 0 m | |
| | | | Transverse Cracking | 5.5m/100m | |
| Michigan (USA) | Asphalt | 5years | Surface distress | Allowable maximum points during each warranty year | Retainment of 10% bid amount |
| | | | Rutting | 0.25 inch | |
| | | | Ride Quality Index | 55 | |
| | Concrete pavement | Unknown | Surface distress | Unknown | Unknown |
| | | | Map cracking | | |
| | | | Joint seal integrity | | |
| Ohio (USA) | Asphalt | 5~7 years | Cracking( width > 6mm) | 150m/160m | Bond: lump sum amount |
| | | | Disintegrated area | None | |
| | | | Flushing | 12 $m^2$/160m | |
| | | | Patching | 28 $m^2$/160m | |
| | | | Rutting | 6mm(5 years), 9.5mm(7 years) | |
| | Chip seal | 3 years | Surface patterns: severe | 40% of segment | Bond: 75% bid amount |
| | | | bleeding/flushing: moderate | 5% of segment length, or 20% localised problems | |
| | | | loss of aggregates: moderate | 10% of segment length, or 20% localised problems | |
| Ontario (Canada) | Micro-surfacing | 2 years | Aggregates loss, raveling and flushing | Unknown | Bond:10% bid amount |

(Source IDOT 1999; MDOT 1995; ODOT 1999a&b)

**Table 5 Maximum number of distress points for any segment during each warranty year**

| Period Year | *Total Distress Points (0.1 mile)* |
|---|---|
| 1 | 3 |
| 2 | 6 |
| 3 | 9 |
| 4 | 12 |
| 5 | 15 |

(Source MDOT 1995)

The remedial works should be performed at no cost to the agency and should be based on the result of pavement distress survey. The remedial work plan should be approved by the agency. The contractor may perform minor routine maintenance during the warranty period. The intent of warranty is to provide a maintenance free pavement for the agency during the warranty period.

The agency should reserve the right to use other contractors to perform emergency action if the problem requires immediate attention for safety reasons. Indiana gives 24 hours to the contractor for emergency works. The contractor is responsible for the payment of all the costs incurred.

Indiana uses A+B biding procedure for warranty asphalt projects. The process takes into account not only the unit prices of the bidder, but also the time within which the bidder proposes to complete the works. During the warranty period, the costs for lane closure will be applied based on peak and non-peak lane closure rates.

### 4.7 Quality control/quality assurance

Under an ideal warranty process, the specification is written in terms of performance directly related to the road users. The construction process and materials will not be specified. The contractor takes full responsibility of quality control/quality assurance. However, in current practices, due to existing knowledge and experience, most agencies prefer to give minimum structure thickness, material properties and quality control limits in the specification. Moreover, the contractor is required to submit their quality control/quality assurance plan to the agency. This situation is expected to be changed when some primary performance relationships are established.

## 5 SUMMARY

The aim of introducing warranty in road construction and maintenance is to achieve better product quality, better management, and potential cost savings. Moreover, the contractor can directly target road users' benefits. Because of limited practices, experience should be shared between agencies in order to successfully promote the approach.

A brief review of various types of specification for road construction and maintenance is given in the paper. Potential benefits, problems associated, and constraints of warranty approach are also investigated. The paper focuses on some key issues in developing a warranty specification for highway construction and maintenance. The issues include criteria for product selection, duration of the warranty, performance indicators and thresholds, quality control/quality assurance, remedial actions, conflict resolution team, warranty bond, payment, etc. A few of currently used warranty specifications are used to identify the issues.

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# CIB Performance Based Building (PeBBu) Thematic Network

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**Summary: The CIB Performance Based Building (PeBBu) Thematic Network is a currently ongoing project. This paper describes the principles of and the assumed impact from Performance Based Building and gives an indication of the international state-of-the-art and main barriers for actual implementation. A summary is given of the overall approach of the PeBBu programme and organization. The main components of the Network are subsequently further elaborated into actual tasks to be performed and deliverables to be produced by the Domain dealing with Construction Materials and Components.**

**Keywords: Performance Based Building, Materials, Components**

## 1 INTRODUCTION

During the period 1998 – 2000 CIB – the International Council for Research and Innovation in Building and Construction - initiated and commissioned various international programmes and R&D projects related to Performance Based Building.

Based on the achievements from those projects CIB proposed in 2000 the establishment of the Thematic Network PeBBu – Performance Based Building. This Network is to elaborate on the activities carried out by CIB, its commissions and its members since it adopted Performance Based Building as a Priority Theme in the CIB Pro-Active Approach in 1998. The PeBBu Network is funded through a Network subsidy within the Growth Programme that is part of the 5th Framework Research Programme of the Commission of the European Communities that was accepted by the EU with a start date of October 1, 2001 to September 30, 2005.

## 2 BACKGROUND:PERFORMANCE BASED BUILDING AND THE CONSTRUCTION INDUSTRY

The prescriptive or deemed-to-satisfy building specifications and codes currently enforced in most countries inhibit both organizational and technological innovation in the construction industry. Performance Based Building strives to overcome this problem by using performance requirements to define a building product's fitness for purpose. Performance Based Building addresses the ends rather than means. This is a strong stimulus for product and process innovation and enhances consumer-orientation.

### 2.1 The construction industry: potential for improvement

In terms of turn over, number of persons employed and social and economic impact the construction industry is one of the largest industries in all developed countries. In 1998 the industry's turnover in the EU was in the magnitude of ECU 700 billion as indicated by the Pederation de L'Industrie Europeenne de la Construction.

For decades in Europe and in most other industrialized parts of the world, the construction market has been primarily supply-driven and consequently the level of customer orientation amongst all professions in the industry traditionally has been rather low. It is to be assumed that that consequently both the level of fitness for use and the level of value for money as perceived by the owners, users and managers of buildings is often substantially less than is the case for many other consumer and investment goods.

Traditionally each building and construction project is dealt with as a prototype. This concerns the design and construction of the project, the technologies to be applied and the collaboration between the various types of companies that each have a role to play in the project. Consequently the level of industrialization and the level of technological innovation in the building and construction industry are rather low as compared to many other industries.

The traditional organization of building and construction projects is strongly fragmented. Especially the separate tasks, responsibilities and liabilities of respectively the architect and the construction firm and the related traditional tendering for construction based on a detailed design with the inclusion of a detailed specification of materials, components and construction technologies, have lead to a situation, in which tendering for the lowest initial price only (as opposed to lowest user costs over time), the almost non-existence of options for technological innovation and optimization for construction firms and a far from optimal communication and information transfer, are regarded as more or less "normal".

The traditional prescriptive or deemed-to-satisfy building regulatory systems, codes and standards as currently in place in almost all industrialized countries, are based upon and enforce the above indicated traditional characteristics of the building and construction industry.

## 2.2 Principles of performance based building

Performance Based Building aims at using performance requirements to define a building or building product's fitness for purpose. Performance Based Building means the orientation on ends rather than means: it describes buildings and building products on the basis of the target performance rather than in terms of solutions and technical specifications. In a situation in which the principles of Performance Based Building are integrally applied it will serve as the basis for communication between all stakeholders in a construction project. In such a situation the respective building regulatory systems and building codes and standards will be performance based and the principles of Performance Based Building will applied on the different interrelated levels of planning (built environment, buildings and building technologies and components) and throughout all phases of the process (brief, design, construction and maintenance)

The actual application of Performance Based Building will have a major impact on the future structure and culture in the building and construction industry and will require fundamentally different business processes in construction firms. The application of Performance Based Building will provide substantial benefits to both the end-user and to the participants in the building process. These benefits stem from a better fitness for use of the building itself, improved communication throughout the building process, better possibilities for innovation in and of the building process leading to optimization of the organization of the building process and production technology, reduced costs and better building quality, increased trade possibilities and specialization in the building industry.

## 2.3 State-of-the-art and main barriers for implementation

In the pro-active application of the principles of Performance Based Building, tenders for the construction of buildings (and civil engineering works) are to be based upon a provisional design plus performance requirements for materials, components and technical systems, as opposed to the traditional tenders that are based upon a detailed design, in which those materials, components and systems are already defined. Performance tendering will allow a construction firm to choose its own materials, components and systems.

Amongst others, the following three major conditions must be fulfilled to make the actual application of Performance Based Building practically possible:

- The language, methods and concepts that are needed to specify those performance requirements as part of the detailed construction specifications, must exist and accepted by the construction professions concerned,
- The methods to establish / measure whether a technical solution fits with the respective performance requirements must exist for materials, components and systems in general and for new, innovative ones in particular,
- There must be performance based building regulatory systems and frameworks, including codes and standards, as a basis for both the detailed specifications and measurement methods.

In certain European countries some experience is already available as concerns the development, implementation and actual application of this approach. An example it the recently introduced Performance Based Building Code in the Netherlands Over the past ten years however various non-European countries, like Japan, the USA, Canada, New Zealand and Australia have begun to develop and implement such systems and have in fact bypassed EU developments. Japan, for instance, is well known for its innovative building technology. Although those techniques will need adaptation to fit the European practice, it is important to learn from the concepts used. North America has established ASTM standards on performance requirements for buildings. North America also has done a substantial amount of research on safety aspects of building. The fire resistance performance is well researched and may offer learning points for the European industry. Canada, Australia and New Zealand have performance based building regulations. Their experiences may also help other European countries.

Also in the area of international standardization for building and construction, initiatives have been taken lately, aimed at re-writing standards based on the principles of PeBBu. ISO/TAG8 aims to stimulate the application of Performance Based Building principles into standards for building and construction. CEN so far has no equivalent.

More recently, Performance Based Building has also been recognized as a major incentive for consumer-oriented building. In Europe, the building market is changing from a supply-oriented market towards a demand-oriented market. The urgent need for housing is almost satisfied; in this sector, the quantitative post-war shortages have been solved to a large extend. However, today the consumer is expressing his requirements far more explicitly than was customary in the past. Consumer-orientation will be the main issue for the building industry in Europe for the years to come. Focusing on end-user requirements will be the main challenge. Finding a language to express the quality of a building in terms understandable to the end-user is a key-issue.

Performance Based Building has the potential to be such language because it tries to relate user requirements to building characteristics and performance.

Additional main components in this broader interpretation are for example:

- A definition of the clients functional and other requirements in terms of performance requirements
- The specification of the designers brief in terms of performance requirements
- Performance-based procurement for both design and construction contracts and often for integrated contracts for construction and – a substantial part of – design (often in the form of design-build procurement)
- New systems to relate costs and (in terms of performance requirements defined) quality
- A performance-based legal framework.

It is to be expected that the actual application of this broader interpretation of Performance Based Building will have a major impact on the future structure of and culture in the building and construction industry and will require fundamentally different business processes in construction firms.

It is also obvious that the application of this broader interpretation of Performance Based Building will not only contribute to stimulating technological innovation, cost optimization and international trade in construction, but explicitly also to achieving more client oriented design and building processes.

In many countries – in Europe for example in the Netherlands, Finland and the UK – a substantial amount of national R&D activities are currently focused on developing operational tools for aspects of this broader interpretation of Performance Based Building.

At the moment, research into Performance Based Building is approached in a rather fragmented way. There are many activities, but these are not coordinated, leading to inefficiency and absence of an integral approach. The “restricted” view, focusing at aspects of the construction process itself has gained substantially more attention than the “broad” view, investigating the contribution of PeBBu to increased consumer-orientation. Some phases, aspects and stakeholders of the building processes have received quite a lot of attention. Examples are structural engineering, fire safety engineering and building project initiation and the construction phase. Other themes and phases have received hardly any attention; for instance use related service aspects, sustainability and communication issues within the building process.

### 2.4 Impact of performance based building: innovation in and of the building process

Although prescriptive criteria are uncomplicated and easy to follow and monitor for architects and contractors, and third parties involved, they also prove to be a significant barrier for innovation, cost optimization and trade:

- Prescriptive regulations block out the application of new, improved or altered products. Those products often do not comply with the solution-based prescription indicating materials, form, size and composition to be used. Performance Based Building would overcome this problem and stimulate innovation, leading to improved building quality.
- The same prescription of solutions hampers the introduction of cheaper products or efficiency measures as the stipulated design also fixes the manufacturing and production process. This blocks, for instance, long-term investments in technological specialization. Performance Based Building would enable optimization of the production process. This may, among other effects, lead to specialization among the parties involved in the building process: it will become worthwhile to invest in the efficiency of a specific production method. This will lead to further industrialization of the building industry, with positive effects for waste control, working conditions and employability. For example performance based tenders for construction will enable construction contractors to immediately realize a substantial optimization of their traditional work processes.
- National prescriptive regulations obstruct international trade: two countries with differing regulations cannot exchange products. Proving the compliance with performance criteria is difficult. Explicit performance based regulations would assist with overcoming this difficulty and increase possibilities for international trade in the building sector.

Performance Based Building will also lead to a reduction of miscommunication throughout the building process, as the required output will be expressed in terms of building performance. This required building performance will serve as a beacon throughout the building process: all agents within the building process will use the same “language” to express the added value of their activities to this required performance.

Performance Based Building asks for a changed attitude and way of working of the parties involved in the building process. The organization of responsibilities may need alteration. The current building process involves fragmented phase-bound and work-type oriented tasks in which each party competes for the maximization of his own profit. Manufacturers, architects and contractors will need to revalue their activities in terms of added quality to the building end-user.

Feedback information from the existing building stock will be necessary to iteratively improve the building process. To enforce the learning capacity of the building industry, the network will stimulate knowledge management: sharing, developing and using knowledge on performance based building between all research and industrial partners involved in building.

### 2.5 Impact of performance based building: improved customer orientation

At the moment consideration of user requirements and the way they change in time, hardly form a significant part in building processes. Buildings are usually designed to fit the first owner or, in many cases, to fit a "fictional" (average) first owner or user. Most attention is being paid at reducing investment costs, while hardly any attention is paid to the way the building will be used throughout the exploitation process and whether or not the building will match the needs of sequential users.

It can be expected that the building market in future will become even more dynamic and the consumer market even more varied. Changing work and living habits ask for new types of living and working accommodation. Various recent developments in the construction market point at a changing market, where the consumer will start defining his requirements and choosing from the available supply. The building sector will have to be able to compete with other consumer goods, such as cars and holidays.

The application of Performance Based Building will enhance consumer-orientation within the building industry, because throughout the building process the explicitly defined user requirements will be the basis for all communication. It will therefore lead to buildings, which better fit the user requirements (both functionally and in terms of costs) and the dynamics in user requirements throughout the exploitation process. It will help parties involved in the building process to take the life span and the performance over time as a key issue in the design and construction process.

A better match between building performance over time and user requirements and more attention for the need for adaptation of buildings, can be expected to decrease maintenance and renovation activities and their impact. Maintenance and renovation activities account for an increasingly large share of the overall volume of output in the building industry (FIEC, 1998). This will lead to resource saving and less building waste from renovation activities required to keep buildings up to date over time.

### 2.6 Overall cost savings after the integral implementation of performance based building

As said already Performance Based Building is a strong stimulus for product and process innovation and enhances consumer-orientation, cost-optimization, and trade possibilities in construction. Performance Based Building is therefore expected to reduce total construction costs by as much as 25% in the long term, as is suggested in table 1.

**Table 1. Estimated savings**

| | Estimated savings in % | |
|---|---|---|
| | Short term | Long term |
| Cost optimisation construction | 2.5 | 3.0 |
| Innovation of products and processes | | 3.0 |
| Specialisation construction firms | | 4.0 |
| Less barriers to international trade | | 2.5 |
| Total process innovation | 2.5 | 12.5 |
| Improved user orientation | 5.0 | 12.5 |
| Total estimated savings in % | 7.5 | 25.0 |

As concerns the above table however it must be realized that the data in this table are in fact not based upon real scientific evidence. They are an indication of a possible magnitude and cannot be more than that as long as international and consensus based methods to measure economic benefits do not exist. Nevertheless, the application of Performance Based Building is anticipated to provide substantial benefits to both the end-user and to the participants in the building process.

## 3 SUMMARY PROGRAMME AND ORGANIZATION NETWORK

### 3.1 Network objectives

The objectives of the PeBBu Network are:

- Stimulation and pro-active facilitation of international dissemination and implementation of Performance Based Building in building and construction practice,

and in that context:

- Maximization of the contribution to this by the international R&D community,

through:

Stimulation and facilitation of the international programming and coordination of research and implementation projects as concerns Performance Based Building as effectively as possible in order to make optimal use of limited available resources and to prevent unnecessary recurrences.

- Stimulation of actual investments in such research and implementation projects.
- Providing the EU Network Members with an optimal access to knowledge and experience as available in non-EU countries in which respective developments have progressed further than in the EU.
- Coordinated dissemination and implementation of results of international research in the area of Performance Based Building.

The Network aims at combining fragmented knowledge in the area of Performance Based Building in order to build a systematic approach towards innovation of the building industry and applying user requirements throughout the building process. From this, white spots and a coherent future research agenda can be derived. End-users, policy makers, building industry and regulatory communities will be closely involved in this development in order to facilitate dissemination and implementation of research results.

The Network will especially stimulate investments in research that may be expected to produce practical recommendations for the adoption and application of Performance Based Building throughout the building industry and in all phases of the building process.

### 3.2 Overall approach

- An infrastructure is established for the programming, coordination and facilitation of research and for the dissemination of research results in the area of Performance Based Building. The main components of this program infrastructure are the following:
- International programming and coordination of research projects in nine scientific domains;
- Involvement of target groups / stakeholders from the start of the programme through three User Platforms for respectively i) buildings owners, users and managers, ii) building and construction industry and iii) the international (pre-) standardisation community;
- Mapping of national and international research as far as related to – aspects of – Performance Based Building;
- Four regional platforms in Europe to act as the bridge to and the initiator of aligned national activities;
- Network Management, including the establishment of a Network Steering Committee, a Technical Committee and a Network Secretariat that among others will be responsible for: i) annual technical and financial reporting to the EU, ii) final report, iii) designated website including among others a newsletter and iv) overall project management.

It is envisaged that at a later stage the following components will be added and that where appropriate separate funding for these will be attracted:

- Three Compendium projects that will serve as a scientific basis for those research projects and that will establish a common framework, a shared language, and the state of the art in terms of research and best practices in the area of i) Validated Models, ii) Economic Benefits and iii) Statements of Requirements
- Development of an advanced IT based facility for the full scale integration of an accessibility to all information in those Compendia and in all other components of the PeBBu Network;
- Various R&D projects related to Performance Based Building, including the about 30 projects that already have been initiated by CIB Task Groups and Working Commissions. The further elaboration of those projects into proposals/request for additional funding has already commenced;
- Various aligned National R&D projects related to Performance Based Building.

### 3.3 Network members

The PeBBu Network includes i) 33 organisations in EU and EU Associated countries and ii) 15 organisations in other countries that have committed to participation under the condition that the required funding will become available.

The Network Consortium consists of nine Principal Contractors. The other initial 25 European organisations in the Network will work as subcontractors to those nine Principal Contractors.

Now that the Network is established and operational, it is open to new membership. New CIB member organizations can submit proposals to the Network Steering Committee. New members accepted will be asked to sign an observer contract that does not provide any funding but allows those members to seek outside funding to support their participation in PeBBu. Details of the PeBBu network and more information can be viewed by accessing the URL http://www.cibworld.nl/pages/ftp/PeBBu_dir/Default.html.

In this context an explicit objective for the Network project Management is to attract new Members from East European and non-European countries and new Members that represent the Building and Construction Industry and other stakeholders as concerns the actual implementation of Performance Based Building.

In principle new PeBBu Members will all be given an "Observer status" and will be linked as such to the Principal Contractor CIBdf; the Technical Coordinating Contractor for the Network.

### 3.4 Nine scientific domains

For each Scientific Domain, in principle, separate international research programming and coordination will take place to ensure internationally accepted prioritizing of research, avoidance of unnecessary doublers, maximum stimulus for international collaboration and a maximum of compatibility between the results from the research projects.

The following nine separate Scientific Domains are distinguished.

IN THE AREA: BUILDING TECHNIQUE

- **Construction Materials and Components**
- **Building Physics**

IN THE AREA: BUILDINGS AND THE BUILT ENVIRONMENT

- **Design of Buildings**
- **Built Environment**

IN THE AREA: BUILDING PROCESS

- **Organization and Management**
- **Legal and Procurement Practices**

IN THE AREA: BUILDING INDUSTRY

- **Regulation**
- **Innovation**
- **Information and Documentation**

At a later stage it may be necessary to combine, split up and/or add such Domains if research developments so require.

## 4 DOMAIN 1 - CONSTRUCTION MATERIALS AND COMPONENTS

As most closely related to the issue of Durability of Building Materials & Components, Domain 1 in the PeBBu project is concerned. This domain has joint CIB leadership by Prof. Christer Sjostrom from Sweden and Dr. Robert Cope from France. For many years a pronounced trend in the area of building and construction technologies, materials and components, towards a performance approach in declaration, assessment and validation of building and construction products, as well as of the functional properties of a constructed works during its lifetime. The trend is pushed by industry and market, but still to a large extent driven by close-to-market and pre- and co-normative research.

### 4.1 Background

In most countries the structural codes has since long moved from simple deemed-to-satisfy approaches to a performance thinking where account is taken of the inherent stochastic variability of material properties and loads. A continuous work is ongoing to fully implement in codes and the resulting engineering the latest development of research and the possibilities offered by new technologies. Computerisation and IT play a decisive role.

As regards the properties of non-load bearing parts of buildings and hence building products the state-of-the-art is still more complex, but the performance road is as evident. As an example, energy conservation and efficiency of building is normally expressed as a performance requirement that then feeds back on the used materials and components. The requirements have then to be declared, assessed and validated for each product. The statistical variability of loads, e.g. degradation factors, and of the material or product properties should ideally be taken into account in a performance assessment. The objective is to in general develop the tools used in engineering design of load bearing and non-load bearing building parts towards the same conceptual approach.

The European Construction Products Directive, CPD (Council Directive 89/106/EEC), which states that the Essential Requirements on constructed works should be met during the intended working life of the building results in essence in a performance requirement on all building products. The life performance of the materials and products has to be assessed and declared.

In the field of Service Life Planning of Buildings and Constructed Assets ISO (TC59/SC14 “Design Life”) is developing performance-based standards. This work was initiated from Europe as a necessary step towards implementing the CPD. Such standards, guiding building design, operation and management as well as the building products producing industry, are intended to assist harmonised design and materials selection in construction, and thus encourage trade liberalisation and in perspective help countries in developing a more safe and sustainable built environment. The main workload lies on completing existing product standards with descriptions of performance based service life assessment and validation methods. There is an increasing international support towards achieving this objective.

### 4.2 State-of-the-art

ISO TC59/SC14 has released two standards (ISO 15686 part 1 and 2) guiding life design of buildings and life prediction procedures of products, respectively.

The bearing idea in life design is that for each building part/product a reference service life is adjusted by factors (environmental load, material quality, workmanship. etc.) describing deviations from the reference situation to the actual building construction scheme, thus reaching a adjusted service life estimate which is then used in the building design. This Factorial Approach needs further development and refinement as regards the factors, the theoretical calculation methods, and the reference life data to be used.

### 4.3 Barriers and opportunities

Of utmost importance to successful development in the Domain, given the above background, is the involvement of the sector industry and other stakeholders in the work. As an example, the main supplier of reference life data on materials and products must be the relevant industry. The involvement of the materials and products producing industry in the process is crucial. The PeBBu network offers obvious possibilities to speed up the process and strengthen the interaction between stakeholders.

### 4.4 Objectives

To foster, through the planned PeBBu workshops and reports, the further development, and anchoring this with sector stakeholders, of the performance concept in the domain of materials and products. This will focus on

- Further development of the Factorial Approach as regards:
    - theoretical and engineering approaches
    - basic knowledge foundation of different factors
    - development of pedagogic application examples
    - test training of practitioners
- exploring and describe the conditions and prerequisites for reference life (performance) data for classes of building materials and components with account of sub-sectorial industry structure,

and thus provide guidelines of pre-standardisation support type on (life) performance of materials and products.

### 4.5 Workplan

The objectives will be met through a step-by-step approach with the purpose to thoroughly describe the pre-conditions to make data on reference service lives and other appurtenant life performance governing data available, i.e. the conditions for establishing reference databases. This will be undertaken by involving major European and international producers of building materials and products, as well as European and international bodies for standards and technical approvals, in the process of establishing information on:

- current policy concerning establishment of durability and service life data
- current format and range of the declaration of durability and service life data, including recommended performance requirement and service environment, that goes with the products.
- current methods to establish such data
- possible availability of today classified data that would be reasonable to make public in case agreement on standards how to report or declare data is made
- opinions on and readiness to meet requirements of ISO 15686-2 as for establishment of data and proposed standards work item on Reference Service Life as for reporting of data
- arguments on the need for corresponding standards at the product level, providing detailed material/product-specific requirements.

In a first step, a questionnaire will be developed and issued addressing the (European, other world regions and national) branch and business organisations of material producers. As an example, on the European scene CEPMC (Council of European Producers of Materials for Construction) is intended to be a major body to be addressed.

Within each country covered by the PeBBu network, a number (3- 5) of the major producers will be selected, supported by the mentioned branch/business organisations, representation of at least all types of high-volume materials/products being ensured. The submitted information will be processed and analysed in relation to the above-mentioned ISO/CEN documents. Workshops together with representatives of the producers will be arranged to discuss the outcome and how to combat obstacles likely to be found. The outcome as well as the findings of the workshops will be documented and reported (see Deliverables below).

The premier international platform for the work to meet the Objectives is the PeBBu Network. In the work process of the Domain Construction Materials and Components will additionally be included, as indicated above, stakeholders and actors from the construction sector and the materials and components producing industry. Examples of main actors to be addressed – in most case contacts on the relevant issues are already well established – are:

– CEPMC and its national “umbrella” organisations and similar organisations in other world regions encompassed by PeBBu
– ISO; relevant ISO TC’s
– CEN; CEN-STAR
– EOTA (European Organisation for Technical Approvals)
– WFTAO (World Federation of Technical Approval Organisations)

### 4.6 Deliverables

The deliverables will in general consist of pre-workshop reports documenting the state-of-the-art as to the issues to be discussed and post-workshop reports developing on the results of the Workshop.

The post-workshop reports will address and develop on the items accounted for under Objectives, i.e.:

– theoretical basis and proposals for engineering approaches of the Factorial Method
– knowledge base of the different factors
– pedagogic application examples showing service life planning of constructed works and parts thereof
– the requirements of and conditions for establishment of generally accessible data bases on (life) performance data

The development of the pedagogic application examples will be performed as test training exercises. The experiences from these exercises will be summarised to a Domain deliverable. Whether to edit the Domain post-workshop reports to one or several Final Deliverable(s) will be discussed during the pace of the PeBBu work.

# The Impact Of Short-life Dwellings On the Total Costs To Sustain Housing

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**Summary: This paper uses a simulation model based on classical population dynamics to make exploratory estimates of the maximum impact of short-life dwellings on the total costs to sustain housing. Data and parameters of long-life dwellings are based on a typical New Zealand dwelling of lightweight timber framing. The size and quality of short-life dwellings are set to be the same as that for long-life dwellings and the service life of short-life dwellings is assumed to be 50 years, the minimum required by the New Zealand Building Code. If the entire simulated housing stock of long-life dwellings is replaced with short-life dwellings, then the total average costs to sustain housing are almost 13% greater when the housing stock expands at 1.5% per year and 24% greater when the housing stock is stationary. Exploratory estimates based on realistic scenarios of depreciation indicate there are no reductions in the national average costs to sustain dwelling services after adjustment for depreciation.**

**Keywords: Dwellings, structural system, service life, refurbishment, national average costs**

## 1 INTRODUCTION

The New Zealand Building Code requires components that contribute to structural stability to have a service life of 50 years (Building Industry Authority 1992). There are two issues here. The first issue is that innovative structural systems need only comply with a service life that falls short of the 90 years average service life of the housing stock as estimated by Johnstone (1994) and well short of the potential service life of most dwellings in the housing stock. Lightweight timber framing has been the predominant structural system used by New Zealand dwellings for over 120 years (Nana 1981) and this structural system has a potential service life of 180 years under New Zealand's climate as illustrated by Kemp House constructed in 1821 (Salmond 1986). Traditional materials used for structural systems such as timber, brick, stone, and concrete have demonstrated an ability to match and exceed a service life of 180 years in other countries (Brand 1997). The second issue is that the most reliable predictor of durability is a successful history of performance. Predictions of the durability of building components based on accelerated aging are fraught with difficulties (Masters 1987). In a recent study of accelerated testing, Rimestad (1998) concludes that field failure data should be used whenever possible to guide the design of accelerated tests. This is not possible with building materials, or combination of materials, that have yet to demonstrate a track record. There is therefore a risk that the projected service life of structural systems based on accelerated aging tests fall short of the minimum service life required by the New Zealand Building Code.

The prudence of allowing structural systems with unproven durability to enter the housing stock and setting a minimum service life that falls well short of the potential service life of most current dwellings needs to be examined for the following reasons. Refurbishment reduces the total costs to sustain housing by extending the service life of dwellings and thereby reducing the replacement rate of the housing stock. In a previous paper, the author estimated that refurbishment of New Zealand dwellings reduces the total costs to sustain housing by at least 15% (Johnstone 2001). The durability of the structural system used by dwellings limits the service life of dwellings and hence restricts potential reductions in total costs. If the durability of the structural systems of new dwellings does not match that of the current housing stock, then future generations will be subject to increased costs to sustain housing.

This paper uses a simulation model based on classical population dynamics to make exploratory estimates of the maximum impact of short-life dwellings on the total costs to sustain housing. Data and parameters of long-life dwellings are based on a typical New Zealand dwelling of lightweight timber framing. The size and quality of short-life dwellings are set to be the same as that for long-life dwellings and the service life of short-life dwellings is assumed to be 50 years, the minimum required by the New Zealand Building Code.

## 2 THE MODEL

### 2.1 Description of simulation model

The simulation model can be visualised as a multi-deck stack where the level of each deck represents the age of a dwelling cohort. A new dwelling cohort enters the top deck of the stack at the start of each time interval and previous dwelling cohorts move progressively down to the next deck. Dwellings are lost from each dwelling cohort over each time interval, the level of which is determined by a probability of loss schedule that forms the first column of a standard life table. The construction of life tables is described in any textbook on population dynamics, for example Keyfitz & Beekman (1984), and Johnstone (2000) examines a selection of analytic, life table, and stock and flow models that can be used to estimate the mortality of housing. New dwelling cohorts continuously replace total dwelling losses of all ages. If the housing stock is stationary and stable, then each new dwelling cohort comprises replacement construction only. When the housing stock undergoes expansion, each new dwelling cohort comprises both replacement construction and new-build construction that adds to the size of the housing stock. The size and quality of each dwelling within the simulated housing stock are set to be homogeneous for the sake of model simplicity. Different typologies of dwellings can be addressed by constructing separate simulation models.

### 2.2 Dwelling services

A physical dwelling is in itself but a means to an end. We are ultimately concerned with the magnitude and quality of the flow of dwelling services that a dwelling can provide. The model in this paper therefore includes the flow of dwelling services provided by housing stock to enable estimates of the costs to sustain each unit of dwelling services. Dwelling services include only those services rendered by improvements to land. The services of land are excluded in order to avoid confounding separate issues of economic depreciation. The quantity of dwelling services supplied are expressed in units of service year equivalents (sye), the dwelling services provided by a dwelling over one year adjusted for economic depreciation. The proxy for benefits can be converted into dollar terms by multiplying by its price.

Standard notation of classical population dynamics is used throughout this paper. Dwelling service years ($L_x$) provided by each dwelling cohort over the age interval $x$ to ($x$ + 1) are given by

$$L_x = l_x - d_x . a_0 \qquad (1)$$

where $l_x$ is the number of dwellings from an original dwelling cohort, $l_0$, which survive to the age $x$, $d_x$ is the number of dwelling losses from a dwelling cohort of age $x$ over the age interval $x$ to ($x$ + 1), and $a_0$ is the average number of dwelling service years provided by dwellings lost over the age interval $x$ to ($x + n$). The value of $a_0$ = ½ when $n$ = 1 gives sufficiently precise results for the purposes of simulating the dynamics of a housing stock.

Stock losses ($d_x$) over each age interval are given by the product of the surviving dwellings at the start of each age interval ($l_x$) and the probability of loss function P($x$, $r$):

$$d_x = l_x . \ \mathrm{P}(x, r) \qquad (2)$$

where $r$ is the annual expansion rate of the housing stock. The probability of loss function P($x$, $r$) is explained in more detail in Section 3.1.

Total dwelling losses of all ages that are lost over the time interval $t$ to ($t$ + 1) are replaced by replacement construction. If the housing stock undergoes expansion, then new dwelling entries include not only replacement construction but also new-build construction. Dwelling entries over the time interval $t$ to ($t$ + 1) are given by

$$l_0 = \sum_{x=0}^{w} d_x + P_{t+1} - P_t = \sum_{x=0}^{w} d_x + (\exp r - 1)P_t = \sum_{x=0}^{w} d_x + (\exp r - 1)\sum_{x=0}^{w} l_x \qquad (3)$$

where $P_t$ is the size of the housing stock at the start of the time interval $t$ to ($t$ + 1) and $P_{t+1}$ is the size of the housing stock at the start of the time interval ($t$ + 1) to ($t$ + 2). The size of the housing stock at time $t$ ($P_t$) is simply the sum of all the dwelling cohorts that are standing at time $t$.

Dwelling services are adjusted for depreciation. The flow of dwelling service year equivalents ($S_e$) provided by a housing stock over the time interval $t$ to ($t$ + 1) is expressed as follows:

$$S_e = \sum_{x=0}^{w} L_x . \mathrm{D}(x) \qquad (4)$$

where D($x$) is an economic depreciation factor that is a function of the age $x$ of dwelling cohorts. This depreciation factor is discussed in more detail in Section 3.2.

The service loss index (SLI) gives the average quality of dwelling services provided by a housing stock expressed in units of dwelling service year equivalents per dwelling per year (sye/dg/yr):

$$\text{SLI} = \frac{S_e}{\sum_{x=0}^{w} L_x} \quad (5)$$

The service life span of a housing stock (ω) is defined here to be that age beyond which less than 0.1% of the oldest dwelling cohort still stands and provides dwelling service. This definition based on Shryock *et al.* (1973) enables sensible comparisons of the service life span of different housing stocks as exceptional dwellings are excluded.

**2.3 Total average costs and national average costs**

The costs to sustain housing are expressed in construction units (cu), the cost to construct a standard New Zealand house as described in Section 3.3. The proxy for costs can be converted into dollar terms by multiplying by its price. The real costs of all forms of construction, maintenance, refurbishment, and demolition work are assumed to remain constant over time. Under this assumption the long run supply curve of the construction industry is perfectly elastic and returns to scale are constant.

The total average costs to sustain housing over the time interval $t$ to $(t + 1)$, $(C_t)$, is the sum of the costs of new-build construction ($C_{\text{new}}$), annual maintenance ($C_{\text{maint}}$), refurbishment ($C_{\text{refurb}}$), demolition ($C_{\text{dem}}$), and replacement construction ($C_{\text{replace}}$) divided by the sum of dwelling service years provided over the time interval:

$$C_{\text{total}} = \frac{C_{\text{new}} + C_{\text{maint}} + C_{\text{refurb}} + C_{\text{dem}} + C_{\text{replace}}}{\sum_{x=0}^{w} L_x} \quad (6)$$

The units of total average costs to sustain housing are in construction units per dwelling per year (cu/dg/yr).

The costs of new-build construction over the time interval $t$ to $(t + 1)$ is given by the number of new-build entries to the housing stock from equation (3):

$$C_{\text{new}} = (\exp r - 1)\sum_{x=0}^{w} l_x . \text{I}_{\text{new}} \quad (7)$$

where $\text{I}_{\text{new}}$ is an index of the costs to construct a dwelling in construction units per dwelling (cu/dg).

For the purposes of this paper, maintenance is defined here as comprising all that work undertaken to retain the provision of essential services such as weather tight shelter, security, lighting, heating, water supply, and waste disposal. The costs of maintenance over the time interval $t$ to $(t + 1)$ is given by:

$$C_{\text{maint}} = \sum_{x=0}^{w} L_x . \text{M}(x) \quad (8)$$

where $\text{M}(x)$ is a function of maintenance in construction units per dwelling service year (cu/sy).

Refurbishment is defined here as the resurfacing or replacement of building components which results in a reversal of the economic depreciation of dwelling services. Refurbishment may take place a number of times over the service life of a dwelling. For the sake of model simplicity, the duration between successive refurbishment cycles for each component is assumed to be regular.

Not all dwellings undergo refurbishment when due. Deferral of refurbishment may be advantageous for budgeting purposes, but can lead to a hastening of physical depreciation if an injudicious selection of deferral has been made. Some forms of physical depreciation are incurable in that the costs of reversing that depreciation by undertaking refurbishment exceed any increase in value to the property. It is therefore in the best rational interests of a property owner not to undertake such refurbishment.

Dwellings depart from the housing stock at all ages (Johnstone 2000). These departures are largely the result of land use succession where increases in the value of land and economic depreciation of improvements make it viable for a property owner to demolish and replace a dwelling or redevelop the site for an alternative use. A greater proportion of each dwelling cohort departs from the housing stock over each successive age intervals (Johnstone 2000) and hence an increasing proportion of dwelling cohorts still standing at the start of each age interval is unlikely to undergo refurbishment over successive age intervals. The proportion of dwellings ($\text{P}_y$) within each cohort that undergo refurbishment at each cycle is therefore set to be the ratio of the dwellings still standing at the end of a refurbishment cycle of duration $z$ to that standing at the start of the cycle

$$\text{P}_y = \frac{l_{y+z}}{l_y} \quad (9)$$

where $y$ is the age at which refurbishment takes place.

The costs of refurbishment over the time interval $t$ to $(t + 1)$ are given by

$$C_{\text{refurb}} \quad = \quad l_a.\mathrm{R}_a.\mathrm{P}_a + \; l_b.\mathrm{R}_b.\mathrm{P}_b \; + ... + \; l_q.\mathrm{R}_q.\mathrm{P}_q \tag{10}$$

where $l_a$, $l_b$, …, $l_q$ are the number of surviving dwellings within a dwelling cohort of ages $y = a, b, \ldots, q$ ; $\mathrm{R}_{\alpha}$, $\mathrm{R}_{\beta, \ldots,}$ $\mathrm{R}_{\alpha}$, $\mathrm{R}_{\beta, \ldots,}$ $\mathrm{R}_{\theta}$ are indexes of the costs to undertake refurbishment at the ages $y = a, b, \ldots, q$ ; and $\mathrm{P}_{\alpha}$, $\mathrm{P}_{\beta, \ldots,}$ $\mathrm{P}_{\theta}$ gives the proportion of each dwellings within a dwelling cohort which undergoes refurbishment at the ages of $y = a, b, \ldots, q$.

The number of departures from the housing stock from equation (3) gives the costs of demolition:

$$C_{\text{dem}} \quad = \quad \sum_{x=0}^{w} d_x.\mathrm{I}_{\text{dem}} \tag{11}$$

where $\mathrm{I}_{\text{dem}}$ is an index of the costs of demolition in construction units per dwelling (cu/dg).

The number of replacement entries to the housing stock from equation (3) gives the costs of replacement construction:

$$C_{\text{replace}} \quad = \quad \sum_{x=0}^{w} d_x.\mathrm{I}_{\text{new}} \tag{12}$$

where $\mathrm{I}_{\text{new}}$ is an index of the costs to replace a dwelling in construction units per dwelling (cu/dg).

The national average costs (NAC) to sustain dwellings services after adjustment for economic depreciation is given by

$$\mathrm{NAC} \quad = \quad \frac{C_{\text{total}}}{\mathrm{SLI}} \tag{13}$$

The units of national average costs to sustain dwelling services are in construction units per dwelling service year equivalents (cu/sye).

## 3 PARAMETERS AND DATA

### 3.1 Probability of loss schedule

Johnstone (1994) established that the mortality of New Zealand housing stock between 1858 and 1980 had been a function not only of age, but also the annual expansion rate ($r$) of the housing stock. The best-fit probability of loss schedule $\mathrm{P}(x,r)$ for New Zealand housing stock is given by a probability of loss schedule which applies for a stationary and stable housing stock and a multiplier function of the annual expansion rate of the housing stock (Johnstone 2000):

$$\mathrm{P}(x,r) \quad = \quad l_x.q_x.(1+78.6r)^{0.70} \tag{14}$$

Each column in a standard life table is a transform of another column (Keyfitz & Beekman 1984). The probability of loss schedule ($q_x$) in the above equation is a transform of the stock losses schedule ($d_x$):

$$d_x = \mathrm{INT}\left[ \frac{l_0.s}{s\sqrt{2p}}\, e^{-\frac{1}{2}[(x-m+0.5)/s]^2} \right] \tag{15}$$

where INT is a function that truncates fractional stock losses to the nearest integer, $l_0$ is initial dwellings entries to a life table at age 0 (the radix of a life table), $s$ is an adjustment factor that ensures the sum of truncated dwelling losses over the service life span of a dwelling cohort equals the initial dwelling entries $l_0$, $s$ *is* the standard deviation of stock losses from a dwelling cohort within a stationary and stable housing stock, and $\mu$ is the mean age of stock losses from a dwelling cohort within a stationary and stable housing stock.

The standard deviation and mean age of stock losses from a cohort of long-life dwellings within a stationary and stable housing stock are set to be 31.08 years and 130 years respectively. The mortality of short-life dwellings is set to be the same as that for long-life dwellings over the age interval 0 to 49 years. Dwellings within each cohort that are still standing at the age of 49 years depart from the housing stock over the age interval 49 to 50 years.

The average annual expansion rate of the New Zealand housing stock has averaged 1.5% per year over the past decade (Statistics New Zealand 1998). Positive net migration has formed the major source of additional household formation because the fertility of the natural population has declined since the 1960s to the extent that the natural population now barely replaces itself.

The average number of persons per household (pph) has declined from over 6 pph at the start of the 20th century to 2.9 pph at the last census in 1996. Further decreases over the next number of decades are likely to be gradual. The annual expansion rates of 1.5%, 1.0%, 0.5%, and 0% per year are selected for the purposes of estimating the maximum impact of short-life dwellings entering the housing stock.

### 3.2 Economic depreciation of dwelling services

An empirical study of the economic depreciation of dwelling services provided by New Zealand housing stock has yet to be carried out. Extensive literature surveys of empirical studies of economic depreciation of dwellings by Malpezzi *et al*. (1987) and Baer (1991) do not provide satisfactory guidelines which can be applied with confidence to New Zealand housing stock as no study estimates the economic depreciation of dwelling services or rent (excluding rent for land) over the full service life of dwellings. A depreciation schedule based on first principles is therefore used instead.

Economic depreciation of dwelling services is due to a combination of physical degradation and obsolescence. A survey of the physical condition of a representative sample of 465 New Zealand dwellings has shown that the average costs of outstanding maintenance (many of these items are refurbishment as defined in this paper) generally rise with house age and the condition of the average house appears constant beyond an age of around 60 years as a consequence of renovation of the older housing stock (Clark *et al.* 2000). Economic depreciation due to physical degradation would therefore decline to a threshold level with fluctuations in the process due to reversals in depreciation after undergoing refurbishment. The rate of economic depreciation due to obsolescence would be gradual over the first years of life of a dwelling, would increase during the middle years, and then diminish in later years due to a vintage effect. The combination of economic depreciation due to physical degradation and obsolescence would follow a fluctuating reversed S-shaped curve that declines to a threshold value. Fluctuations in economic depreciation are ignored for the sake of simplification and a reversed S-shaped curve function is taken to be a likely schedule of economic depreciation. Reversed S-shaped curve depreciation is based on the logistic curve

$$\mathrm{Q}(x) = \frac{\mathrm{K}}{1 + ce^{-r_m x}}, \quad c = \frac{\mathrm{K}}{\mathrm{Q}(0)} - 1 \tag{16}$$

and

$$\mathrm{D}(x) = \frac{\mathrm{P\text{-}Q}(x)}{\mathrm{P\text{-}Q}(0)} \quad \text{for} \quad \mathrm{P} \geq \mathrm{Q}(x) \tag{17}$$

The units of D($x$) are dimensionless and $0 \leq \mathrm{D}(x) \leq 1$. For convenience, P = 10,010, Q(0) = 10, and $r_m$ is varied.

In the absence of empirical data on the value of the dwellings services provided by the oldest dwellings in the housing stock immediately prior to departure from the housing stock, the threshold value of the depreciation function D(ω) is set to a value of 0.50 and 0.25. The same schedules of depreciation are set to apply for long-life and short-life dwellings.

The probability of loss of dwellings increases as the expansion rate of the housing stock increases and there is a corresponding decrease in the average service life and service life span of the housing stock. A decline in the value of dwellings services and hence a decline in capital value would hasten departures from the housing stock. An increase in the rate of economic depreciation of dwelling services would therefore accompany an increase in the probability of loss of dwellings.

Straight-line depreciation is included for comparison:

$$\begin{aligned} \mathrm{D}(x) &= 1 - \frac{(x+1)}{w_p} \quad \text{when} \quad x < w_p \\ \mathrm{D}(x) &= 0 \quad \text{when} \quad x \geq w_p \end{aligned} \tag{18}$$

where $\omega_p$ is that age at which D(ω) = 0.50 and D(ω) = 0.25.

### 3.3 Costing data

The costs of new-build and replacement construction of long-life dwellings are based on the National Modal House (NZIV 1996) which comprises a typical New Zealand dwelling of lightweight timber framed construction with a floor area of 100 $m^2$. The floor and sub-floor structure is particleboard on timber joists, bearers, and timber piles. The wall cladding is fibre cement planks, windows and exterior doors are aluminium, and the roofing is galvanised mild steel. Interior linings are gypsum board. The costs of the Modal House are $94,110.40 in June 1997 New Zealand dollars. This paper includes vinyl flooring (10 $m^2$) in the kitchen and a 3-coat polyurethane floor finish in all other areas which brings the total average costs of a long-life dwelling to NZ$95,155.40.

Dufaur (2001) has compared the costs of five alternative structural systems to lightweight timber framing as used in New Zealand. Two of these alternatives, lightweight steel framing and wood-fibre panels, use non-traditional building materials which have yet to demonstrate a service life which matches the 90 year average service life of the housing stock. Although cost savings of up to 3% can be achieved by using either lightweight steel framing or wood-fibre panels to construct a dwelling to the same size and quality as the National Modal House (Dufaur 2001), the costs of short-life dwellings are taken to be the same as that for long-life dwellings because these cost savings fall within the margin of error. The possibility of future reductions in costs of lightweight steel framing and wood-fibre-panels are also examined.

Estimates of the costs of maintenance are based on the maintenance records of 25 New Zealand Housing Corporation dwellings that date back between 24 and 39 years prior to 1988. Table 1 sets out the average annual costs of maintenance based on these records. The total annual costs of maintenance ($269.91) are set to apply for both long-life and short-life dwellings and these costs are assumed to remain constant over the full service life of dwellings.

Table 2 lists the cycles and costs of refurbishment including removal and disposal of existing components and making good to collateral damage in the process. The cycles and costs apply for both long-life and short-life dwellings with exceptions as listed. Refurbishment of short-life dwellings is not undertaken when the remaining service life of dwellings is shorter than the refurbishment cycle *z*. The replacement of component costs are based on the schedule of the National Modal House (NZIV 1996) and pricing data provided by Rawlinsons Group (1997). The cost of demolition and disposal of both short-life and long-life dwellings is $1,700. Recycling of building materials is not examined in this paper.

**Table 1. Average annual costs of maintenance**

| *Item* | *Cost (1997 NZ$)* |
|---|---|
| Electric range: repairs | 26.20 |
| Hot water cylinder: repairs | 20.52 |
| Electrical: outlets, lighting, meter board, minor wiring | 27.01 |
| Taps: washers, replacement | 10.58 |
| Waste pipe work: clear blockages, repair soil pipe junctions | 13.09 |
| Water supply | 15.90 |
| Drainage system: clear blockages, repair soil line | 23.18 |
| Spouting and downpipes: clean out, repair | 26.24 |
| Flashings: repair | 3.24 |
| Hardware: door locks, window catches | 6.75 |
| Glazing: repairs to windows | 4.08 |
| Miscellaneous | 56.63 |
| Unspecified due to illegibility or inadequacy of entry on card | 36.49 |
| Total | 269.91 |

**Table 2. Costs of refurbishment**

| *Component* | *Short Life Dwellings* | *Propn (%)* | *Cost (1997 NZ$)* | *Cycle z (years)* | *Source of cycle and proportion* |
|---|---|---|---|---|---|
| Substructure | no | 15 | 1,326.87 | 40 | Tucker & Rahilly (1990) |
| Wall framing | no | 15 | 1,246.02 | 40 | Tucker & Rahilly (1990) |
| External cladding & trim | no | 100 | 5,611.42 | 50 | Page (1997) |
| Internal linings & trim | no | 30 | 2,138.27 | 40 | Tucker & Rahilly (1990) |
| Alumin. Windows & doors | no | 100 | 7,975.61 | 40 | NBA Consultants (1985) |
| Fittings: kitchen, bathroom | yes | 100 | 8,307.88 | 25 | NBA Consultants (1985) |
| Combustion heater | no | 100 | 3,472.78 | 40 | NBA Consultants (1985) |
| Roofing | no | 100 | 4,530.88 | 50 | Page (1997) |
| PVC spouting, downpipes | yes | 100 | 1,128.60 | 20 | NBA Consultants (1985) |
| Plumbing piping & traps | no | 100 | 1,812.82 | 40 | NBA Consultants (1985) |
| Plumbing fittings | no | 100 | 2,825.88 | 40 | NBA Consultants (1985) |
| Electrical: wiring | no | 50 | 1,452.42 | 40 | Tucker & Rahilly (1990) |
| Electrical: stove & HWC | yes | 100 | 1,477.80 | 25 | NBA Consultants (1985) |
| Prep & painting interior | yes | 100 | 3,575.09 | 8 | Tucker & Rahilly (1990) |
| Prep & painting of roof | yes | 100 | 1,117.80 | 7 | Page (1997) |
| Prep& painting of cladding | yes | 100 | 1,705.72 | 9 | Page (1997) |
| Floor covering: vinyl sheet | no | 100 | 360.00 | 30 | NBA Consultants (1985) |
| Polyurethane floor finish | yes | 100 | 1,027.40 | 10 | NBA Consultants (1985) |

# 4 RESULTS

Housing stocks are comprised entirely of either long-life or short-life dwellings. Table 3 summarises the results for housing stocks that are subject to reversed S-shaped curve economic depreciation with a threshold depreciation of D(ω) = 0.50. The total average costs of short-life dwellings (cu/dg/yr) are 23.9% greater than that for long-life dwellings when the housing stock is stationary and are 12.7% greater when the expansion rate $r$ = 1.5%. Refurbishment costs of short-life dwellings are smaller, but replacement costs are much greater for all expansion rates. Refurbishment of long-life dwellings forms a larger proportion of total average costs and this proportion decreases by a greater extent as the expansion rate increases (60.0% when $r$ = 0% and 36.9% when $r$ = 1.5%) compared to short-life dwellings (29.4% when $r$ = 0% and 23.1% when $r$ = 1.5%).

The national average costs to sustain short-life dwellings (cu/sye) are 4.4% greater than that for long-life dwellings when the housing stock is stationary and are 5.8% greater when $r$ = 0.5%. Increases in national average costs peak at 7.8% when $r$ = 0.5%. The increases in national average costs are substantially less than increases in total average costs at low expansion rates due to short-life dwellings undergoing much less depreciation over a shorter service life. The mean ages of short-life and long life dwellings are 24.7 and 68.1 years respectively when the housing stock is stationary compared to 21.7 and 36.3 years when $r$ = 1.5%.

Table 4 tabulates the national average costs for housing stocks that are subject to straight-line curve economic depreciation with a threshold depreciation of D(ω) = 0.50. Increases in national average costs to sustain short-life dwellings are greater when subject to straight-line depreciation because short-life dwellings undergo relatively greater depreciation. The national average costs to sustain short-life dwellings (cu/sye) are 7.9% greater than that for long-life dwellings when the housing stock is stationary and are 6.6% greater when $r$ = 1.5%. Increases in national average costs peak at 9.3% when $r$ = 0.5%.

**Table 3. Results for reversed S-shaped curve depreciation D(w) = 0.50**

| | *R = 0%* | | *r = 0.5%* | | *r = 1.0%* | | *r = 1.5%* | |
|---|---|---|---|---|---|---|---|---|
| | *(value)* | *(%)* | *(value)* | *(%)* | *(value)* | *(%)* | *(value)* | *(%)* |
| *Long-life dwellings* | | | | | | | | |
| Average service life (years) | 129.3 | | 104.0 | | 97.2 | | 93.1 | |
| Service life span (years) | 180.0 | | 169.0 | | 155.0 | | 148.0 | |
| Mean age (years) | 68.1 | | 49.0 | | 41.6 | | 36.3 | |
| New-build costs (cu/dg/yr) | 0.0000 | 0.0 | 0.0050 | 16.7 | 0.0101 | 30.1 | 0.0152 | 41.0 |
| Maintenance costs (cu/dg/yr) | 0.0028 | 10.6 | 0.0028 | 9.4 | 0.0028 | 8.5 | 0.0028 | 7.7 |
| Refurbishment costs (cu/dg/yr) | 0.0161 | 60.0 | 0.0147 | 48.8 | 0.0141 | 42.2 | 0.0136 | 36.9 |
| Demolition costs (cu/dg/yr) | 0.0001 | 0.5 | 0.0001 | 0.4 | 0.0001 | 0.3 | 0.0001 | 0.3 |
| Replacement costs (cu/dg/yr) | 0.0077 | 28.9 | 0.0075 | 24.7 | 0.0063 | 18.9 | 0.0053 | 14.2 |
| Total average costs (cu/dg/yr) | 0.0268 | | 0.0302 | | 0.0335 | | 0.0370 | |
| Service loss index (sye/dg/yr) | 0.837 | | 0.901 | | 0.914 | | 0.929 | |
| National average costs (cu/sye) | 0.0320 | | 0.0335 | | 0.0366 | | 0.0398 | |
| *Short-life dwellings* | | | | | | | | |
| Average service life (years) | 49.5 | | 49.4 | | 49.3 | | 49.3 | |
| Service life span (years) | 50.0 | | 50.0 | | 50.0 | | 50.0 | |
| Mean age (years) | 24.7 | | 23.7 | | 22.7 | | 21.7 | |
| New-build costs (cu/dg/yr) | 0.0000 | 0.0 | 0.0051 | 14.1 | 0.0101 | 26.2 | 0.0152 | 36.5 |
| Maintenance costs (cu/dg/yr) | 0.0028 | 8.6 | 0.0028 | 7.9 | 0.0028 | 7.3 | 0.0028 | 6.8 |
| Refurbishment costs (cu/dg/yr) | 0.0097 | 29.4 | 0.0097 | 27.2 | 0.0097 | 25.1 | 0.0096 | 23.1 |
| Demolition costs (cu/dg/yr) | 0.0004 | 1.1 | 0.0003 | 0.9 | 0.0003 | 0.7 | 0.0002 | 0.6 |
| Replacement costs (cu/dg/yr) | 0.0202 | 61.0 | 0.0179 | 49.9 | 0.0157 | 40.6 | 0.0137 | 32.9 |
| Total average costs (cu/dg/yr) | 0.0332 | | 0.0358 | | 0.0386 | | 0.0417 | |
| Increase in total average costs (%) | 23.9 | | 18.5 | | 15.2 | | 12.7 | |
| Service loss index (sye/dg/yr) | 0.993 | | 0.992 | | 0.990 | | 0.990 | |
| National average costs (cu/sye) | 0.0334 | | 0.0361 | | 0.0390 | | 0.0421 | |
| Increases in NAC (%) | 4.4 | | 7.8 | | 6.6 | | 5.8 | |

**Table 4. Results for straight-line curve depreciation D(w) = 0.50**

| | *r = 0%* | *r = 0.5%* | *r = 1.0%* | *r = 1.5%* |
|---|---|---|---|---|
| *Long-life dwellings* | | | | |
| Service loss index (sye/dg/yr) | 0.811 | 0.854 | 0.865 | 0.876 |
| National average costs (cu/sye) | 0.0331 | 0.0353 | 0.0387 | 0.0422 |
| *Short-life dwellings* | | | | |
| Service loss index (sye/dg/yr) | 0.930 | 0.929 | 0.926 | 0.926 |
| National average costs (cu/sye) | 0.0357 | 0.0386 | 0.0417 | 0.0450 |
| Increase in national average costs (%) | 7.9 | 9.3 | 7.8 | 6.6 |

Table 5 tabulates the national average costs for housing stocks that are subject to reversed S-shaped and straight-line curve economic depreciation with a threshold depreciation of D(ω) = 0.25. Increases in national average costs (cu/sye) for short-life dwellings are substantially reduced to the extent that there are savings of 5.4% and 1.1% when the housing stock is stationary and subject to reversed S-shaped curve and straight-line curve depreciation respectively. Increases in national average costs for short-life dwellings are otherwise modest peaking at 2.5% and 3.9% at $r$ = 0.5% when subject to reversed S-shaped curve and straight-line curve depreciation respectively.

**Table 5. Results for reversed S-shaped and straight-line curve depreciation D(w) = 0.25**

| | *r = 0%* | *r = 0.5%* | *r = 1.0%* | *r = 1.5%* |
|---|---|---|---|---|
| *Reversed S-shaped curve depreciation* | | | | |
| Long-life dwellings national average costs (cu/sye) | 0.0354 | 0.0353 | 0.0383 | 0.0413 |
| Short-life dwellings national average costs (cu/sye) | 0.0335 | 0.0362 | 0.0391 | 0.0422 |
| Increase in national average costs (%) | -5.4 | 2.5 | 2.1 | 2.2 |
| *Straight-line curve depreciation* | | | | |
| Long-life dwellings national average costs (cu/sye) | 0.0374 | 0.0386 | 0.0420 | 0.0454 |
| Short-life dwellings national average costs (cu/sye) | 0.0370 | 0.0401 | 0.0435 | 0.0469 |
| Increase in national average costs (%) | -1.1 | 3.9 | 3.6 | 3.3 |

Some structural systems, for example lightweight steel framing, are direct substitutes for lightweight timber framing in that interior linings and exterior cladding systems can be the same. The costs of lightweight timber wall and roof framing are $13,181.69 or 13.9% cost of the total average costs of $95,155.40 for the National Modal House including floor finishes. If the costs of a direct substitute short-life structural system for the walls and roof are 75% and 50% of that for lightweight timber framing, then the costs of new-build and replacement construction would be $91,859.98 or 0.9654 construction units and $88,564.56 or 0.9307 construction units respectively. Table 6 tabulates the increases in total average costs (cu/dg/yr) and national average costs (cu/sye) for short-life dwellings when the costs of structural systems for the walls and roof are 75% and 50% of that for long-life dwellings. The housing stocks are subject to reversed S-shaped curve economic depreciation with a threshold depreciation of D(ω) = 0.50.

**Table 6. Results for reduced costs short-life dwellings**

| | *r = 0%* | *r = 0.5%* | *r = 1.0%* | *r = 1.5%* |
|---|---|---|---|---|
| *Cost of wall and roof structural system 75%* | | | | |
| Total average costs (cu/dg/yr) | 0.0325 | 0.0352 | 0.0381 | 0.0412 |
| Increase in total average costs (%) | 21.3 | 16.6 | 13.7 | 11.4 |
| Increase in national average costs (%) | 2.2 | 6.0 | 5.2 | 4.5 |
| *Cost of wall and roof structural system 50%* | | | | |
| Total average costs (cu/dg/yr) | 0.0318 | 0.0346 | 0.0376 | 0.0407 |
| Increase in total average costs (%) | 18.7 | 14.6 | 12.2 | 10.0 |
| Increase in national average costs (%) | 0.0 | 4.2 | 3.6 | 3.3 |

The total average costs of short-life dwellings (cu/dg/yr) are 21.3% and 18.7% greater than that for long-life dwellings when the housing stock is stationary and the costs of the wall and roof structural systems are respectively 75% and 50% of that for long-life dwellings. These increases in total average costs decline to 11.4% and 10.0% respectively when $r$ = 1.5%.

The costs to construct a short-life dwelling to the same size and quality as that of the National Modal House would need to be between 68.5% to 65.5% of that for long-life dwellings in order for the total average costs (cu/dg/yr) to be the same as that for long-life dwellings when $0 \leq r \leq 1.5$.

Increases in national (cu/sye) costs decline to a peak of 6.0% (*r* = 0.5%) and a low of 2.2% (*r* = 0%) when the costs of the wall and roof structural system is 75% of that for long-life dwellings and decline to a peak of 4.2% (*r* = 0.5%) and a low of 0.0% (*r* = 0%) when the costs of the wall and roof structural system is 50% of that for long-life dwellings.

## 5 CONCLUSIONS

This paper uses a dynamic simulation model to make exploratory estimates of the maximum impact of short-life dwellings on the total costs to sustain housing. Long-life dwellings are based on a typical New Zealand dwelling of lightweight timber framing. Short-life dwellings of the same size and quality are assumed to have a service life of 50 years, the minimum required by the New Zealand Building Code. If the entire simulated housing stock of long-life dwellings is replaced with short-life dwellings, then the total average costs to sustain housing are almost 13% greater when the housing stock expands at 1.5% per year. This increase is equivalent to an additional expenditure of $600 million on New Zealand housing in 2001. Total average costs are 24% greater when the housing stock is stationary and are the same as that for long-life dwellings when the costs to construct short-life dwellings are 65% of that to construct long-life dwellings. The quality of dwelling services provided by a short-life, and therefore younger, housing stock is higher than that provided by long-life housing stock. Nonetheless, exploratory estimates based on realistic scenarios of depreciation indicate there are no reductions in the national average costs to sustain dwelling services after adjustment for depreciation when the costs to construct short-life dwellings are the same as that to construct long-life dwellings.

The results of this paper signal that it is imprudent to allow short-life dwellings to enter the New Zealand housing stock unless the costs to construct short-life dwellings are substantially less than the costs to construct long-life dwellings. In addition to significant increases in costs to sustain housing, the penalties of short-life dwellings entering the housing stock include increases in environmental pollution from manufacturing processes, waste products from demolition, and $CO_2$ contributions to the atmosphere due to activities by the construction industry. The full extent of these penalties would be carried by future generations, as it would take one service life span for the entire housing stock to be replaced by short-life dwellings. Some may therefore dismiss the need to restrict the entry of short-life dwellings into the housing stock. Under the principles of intergenerational justice, we should do unto the next generation as we would have the previous generation do unto us. Intergenerational justice would be assured under Rawls' (1973) veil of ignorance where no one knows which generation they belong to when they make decisions which have an impact on future generations.

## 6 ACKNOWLEDGMENTS

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# The Sealing Of Deep-Seated Swiss Alpine Railway Tunnels New Evaluation Procedure For Waterproofing Systems

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**Summary**: **A comprehensive evaluation procedure for the sealing and drainage of the two long tunnels through the Swiss alps is performed based on an order by AlpTransit Gotthard AG and BLS AlpTransit AG. Special influences such as high geothermal heat, high pressure of mountain mass, construction, and high expectations for the service life-time – 100 years – had to be considered. Compared with near-surface tunnels, the thermoplastic polymeric materials typically used at present are more highly stressed. Polymeric products combined to waterproofing systems (waterproofing membranes: PE, PE-Cop, PVC-P, drainage materials: PE, PP, PA, PES) were submitted by the applicants and tested in a 24 months program for their ageing resistance. Existing test methods were complemented by new procedures, e.g., by resistance at elevated temperatures and in oxygen-enriched water at elevated temperatures, respectively, compression creep tests between rough surfaces, behaviour under combined lateral compression and horizontal shear, and installation tests including construction of the conrete support shell. The most important results of the four test modules with preliminary conclusions will be detailed. The installation tests showed that the waterproofing membranes develop regular folds with small radii of curvature during construction and thus locally lead to high strains in the membranes. Since most products did not meet certain requirements, optimised systems are now re-evaluted in a shortened procedure.**

**Keywords. Tunnel construction, polymeric waterproofing systems, evaluation, long-term behavior, ageing resistance**

## 1 INTRODUCTION - DESCRIPTION OF PROBLEM

The two new railway connections from North to South in Switzerland, the Gotthard and Lötschberg base tunnels, are being built with one track each per gallery as double-shell tunnels. Because of the large mountain cover, the waterproofing system (WPS) between the relatively rough shotcrete outer shell and the concrete inner shell should continuously drain the attracted mountain water, protect the concrete construction against water, and locally transfer high compressive loads onto the concrete support structure. At the base, the large mountain cover of up to 2'500 m also results in rock temperatures of the order of 45° C due to geothermal effects. These conditions thus apply to the intruding water that is mostly alcaline, but may be acidic in some areas. The expected service life-time is 100 years, with no major repairs being necessary within 50 years. All waterproofing and draining materials known today have not been designed for nor ever been tested under such extreme conditions. The known standards do not contain criteria or requirements for such loads. Results of long-term tests were not available at the time and practical experience existed only for relatively short periods.

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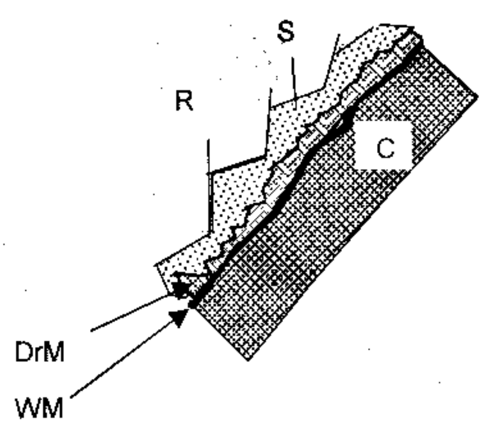


**Figure 1: Detail of the tunnel wall: Concrete support C, waterproofing membrane WM, drainage material DrM, shotcrete surface S, mountain/rock R**

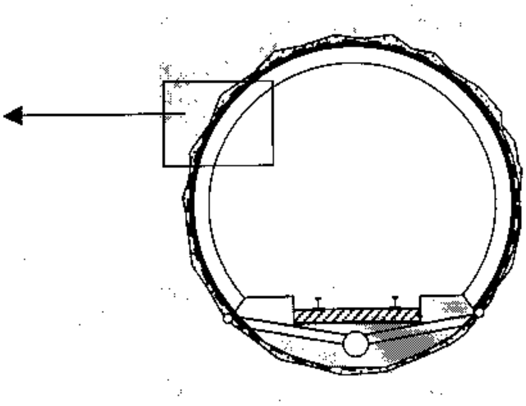

**Figure 2: Schematic cross-section of a deep-seated tunnel indicating position of the waterproofing system, the tunnel is drained pressure-less.**

## 2 EVALUATION PROCEDURE

In order to effectively take into account the special conditions (mountain, choice of construction, and other boundary conditions imposed by the builders), a special task force together with the assigners developed, invited for tender, and realised a multi-part procedure for the evaluation of the waterproofing system consisting of waterproofing membrane and drainage material (E. Basler+Partner 1998). This procedure comprised:

- Ageing behaviour of the different waterproofing and drainage materials
- Behaviour of the WPS under compression creep load
- Compression/shear behaviour of the WPS
- Suitability for installation
- Synthesis

### 2.1 Ageing of the individual components

In this part of the evaluation, the components of the WPS are exposed for 24 months without mechanical loading to the following media: water circulated at temperatures of 23°, 45°, and 70° C, alcaline and acidic water at 50° C, oxygen-enriched water at 70° C and 3 bar pressure (test equipment shown in Figs 3 and 4), and, by burying the specimens, in an environment with aerobic and anaerobic microorganisms. At 5 points during the storage, the waterproofing membranes are tested for mass changes, longitudinal and transverse change of dimensions, mechanical puncture strength (dart impact on WPS placed on rigid support), and the drainage materials for mass changes, transmissivity within the plane, and punching resistance against a drop cone. An additional series of tests such as, e.g., tensile, thermomechanical and thermoanalytical after 3, 6, 12 and 24 months is used to determine the respective properties for a sufficiently complete description of the ageing processes.

### 2.2 Behaviour of the WPS under Compression Creep Load

In a separate compression test during 24 months, the WPS are stored at 0.4 MPa in alcaline water at 50° C between a planar and a strongly corrugated ripple plate (Fig. 5). The surface roughness Rt determined using the sand-filling procedure is 4.5 mm. This ripple structure consists of 169 truncated pyramids with a square base area of 256 mm$^2$, a height of 7 mm and a top area of 4 mm$^2$. Figure 6 shows the experimental arrangement for the WPS. The decrease of the thickness of the waterproofing system, the impermeability,

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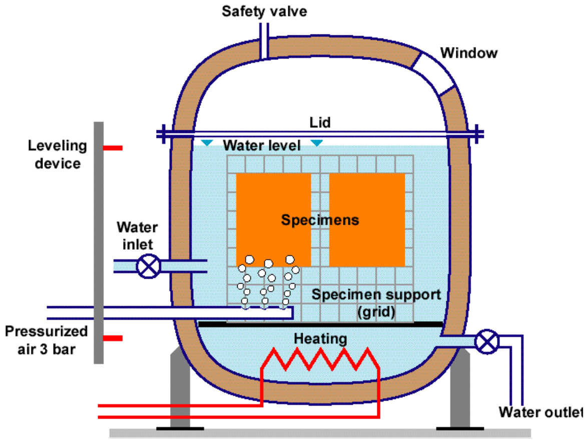
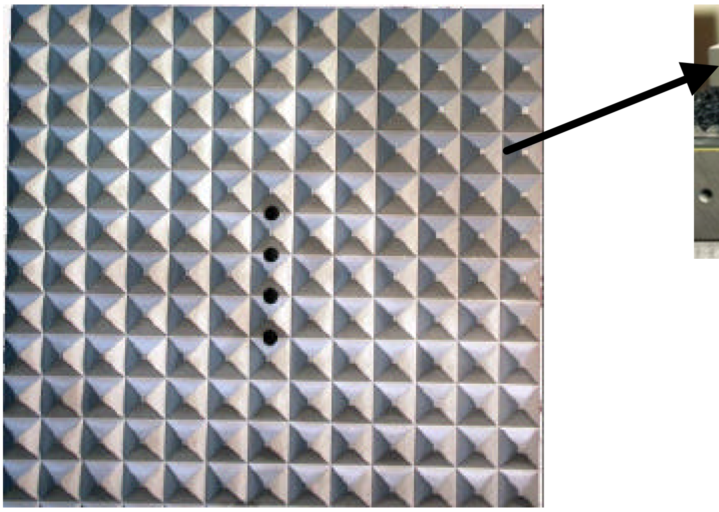



**Figure 3: Pressure vessel for ageing in oxygen-enriched water at elevated temperature and pressure**

**Figure 4: Schematic of the pressure vessel with specimens (vertical arrangement immersed in water)**

the drainage capacity, the visual appearance and the interaction betweeen waterproofing membrane and drainage material are periodically recorded.



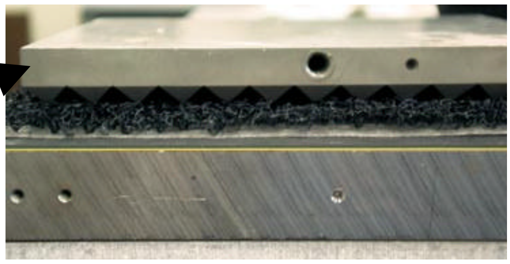


**Figure 5: Set-up of the compression creep test: from top to bottom ripple plate, drainage mat, waterproofing membrane, planar steel plate**

**Figure 6: Example of the ripple plate (surface towards drainage material) with drainage water outlets**

## 2.3 Compression/Shear Behaviour of the WPS

Four separate test modules are used to evaluate the short- and long-term behaviour of the WPS under the combined effects of compression (up to 2 MPa) and shear at 45° C for a 1.5 $m^2$ size specimen between the rough shotcrete surface (average surface roughness Rt 3.1 mm) and the concrete compressive support surface. The waterproofing function, the drainage capacity, and the deformation behaviour of the systems set-up are evaluated. Because of the novelty and complexity of this test it was necessary to specially design, test, and validate a suitable test set-up. The reproducibility of the test results was assured by casting one template each of an original shotcrete and concrete surface. A pipeline system inserted into the pressure plates allows heating the contact surfaces to up to 70° C. With an additional pipeline system, drainage water can be circulated into a water distribution groove placed at the center of the top plate from where it will flow horizontally through the WPS (Fig. 7). Drinking water, filtered at the inlet to remove eventual suspended particles (> 10 µm) was used. The water flowing out along the sides is collected and pumped back into the circuit. The drainage capability is measured by determining the pressure necessary for a constant water throughput of 10 l/min.

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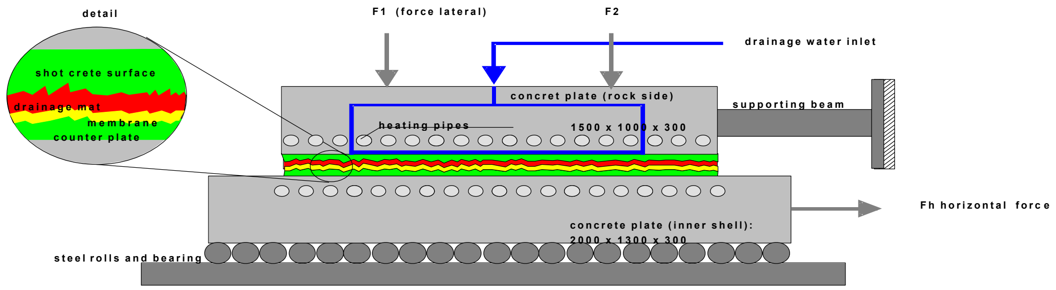


**Figure 7: Schematic cross-section of the compression/shear set-up with heating and drainage capability, the top plate (fixed) corresponds to the shotcrete surface of the outer tunnel shell**

### 2.4 Suitability of Installation

In an experimental gallery with a 8 m high shotcrete arch of the vault, each waterprofing system (WPS) is used to seal a shotcrete surface area of 70 $m^2$ with a defined surface roughness under real conditions (humid-wet at elevated temperatures of the shotcrete surface) including construction of the concrete support shell. The type of fixation, the number of fixations, the joint type and the properties of the seams are checked as indicators for the WPS. After completion of the construction, the water flow and drainage capability, respectively, are measured by injecting a defined amount of water through a piping system into the spherical cap behind the waterproofing membrane. Later, the concrete support shell is removed and the exposed WPS is inspected for damage/bruises and impermeability, respectively, for fold-free installation and other irregularities.

## 3 MATERIALS TESTED

The applicants, i.e., consortia of manufacturers, suppliers, and installation companies, submitted 14 different combinations of WPS (Table 1) for the two year evaluation period. Those combinations were considered optimal for use in the alpine tunnels. The considerable differences between the submitted WPS resulted from different weight attributed to the various conditions (waterproofing, drainage function, pressure transfer, and fire resistance during installation) that had to be fulfilled simultaneously. These requirements were interpreted quite differently by the applicants.

**Table 1: Types of Materials submitted for the two year evaluation procedure**

| *Aspect* | *Unit* | *Waterproofing Membranes* | *Drainage Materials* |
|---|---|---|---|
| Number | Piece | 10 | 8 |
| Material Type | - | PVC-P, PE, PO-Copolymers (EVA) | PE-HD, PA, PES, PP |
| Structure | - | mostly smooth surfaces, partially with stamping, with inserted non-woven glass-fabric, textile reinforcement | dimpled sheet<br>net<br>randomly oriented mat<br>non-woven geotextiles, dense, rough |
| Thickness | mm | 2.1 – 3.3 | 6 to 18 |
| Areal Weight | kg/m2 | 2.1 - 3.0 | 0.55 – 1.9 |
| Width of Rolls | m | 2 | 1 - 2 |
| Type of Joint | - | hot-air welding | no joining, overlapping |

Prior to acceptance in the evaluation, the applicants had to proove conformity with existing standards/guidelines, such as Sia V280 for ground-water sealing (Sia 1996) and Handbook of Geotextiles (SVG). Since these standards apply only to materials after manufacturing, some requirements were specifically increased, in particular mechanical puncture strength for the waterproofing membranes, and transmissivity and fire classification for the drainage materials.

## 4 RESULTS

### 4.1 Ageing Behaviour

After 6 months, the results were relatively heterogeneous, but after 12 months, differences in quality between the types of materials used for the waterproofing membranes and the drainage became apparent. Between 12 and 24 months, these trends were reinforced. In general, the waterproofing membranes prooved to be more resistant than the drainage materials even

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though some products made from polyolefin-copolymers (PO-C) decomposed during the exposure to hot, oxygen-enriched water. Waterproofing membranes made from plasticized PVC, as expected, reacted by loss of plasticiser most strongly to alcaline water and by increasing loss of plasticiser under exposure to aerobic microorganisms. Some materials made of PO-Copolymers reacted when exposed to alcaline water. The evaluated drainage materials generally reacted more sensitively to both acidic and acaline exposure. Some products made from polyamide (PA) and saturated polyesters (PES) decomposed in acidic water at 50° C. In hot water at 70° C, these materials embrittled even after 6 months, at 45° C between 12 and 24 months. Dimpled sheets yielded relatively minor changes.

### 4.2 Behaviour under Compression Creep Load

Under this type of loading, the drainage materials yielded considerable deformations and the ripple plate intruded deeply through the draining layer into the waterproofing membrane. From the compression-deformation curves (Diagram 1) the thickness of the total WPS at the load points is reduced to 0.8 mm. No WPS, however, was punctured under this creep load. Even an additional 20% strain after the test did not show disadvantageous behaviour at the contact points. In a few cases, the drainage stuck to the waterproofing membrane. Points of high stresses, e.g., at the edge of the nop, occasionally yielded first signs of stress induced cracks that increased in length between 12 and 24 months.

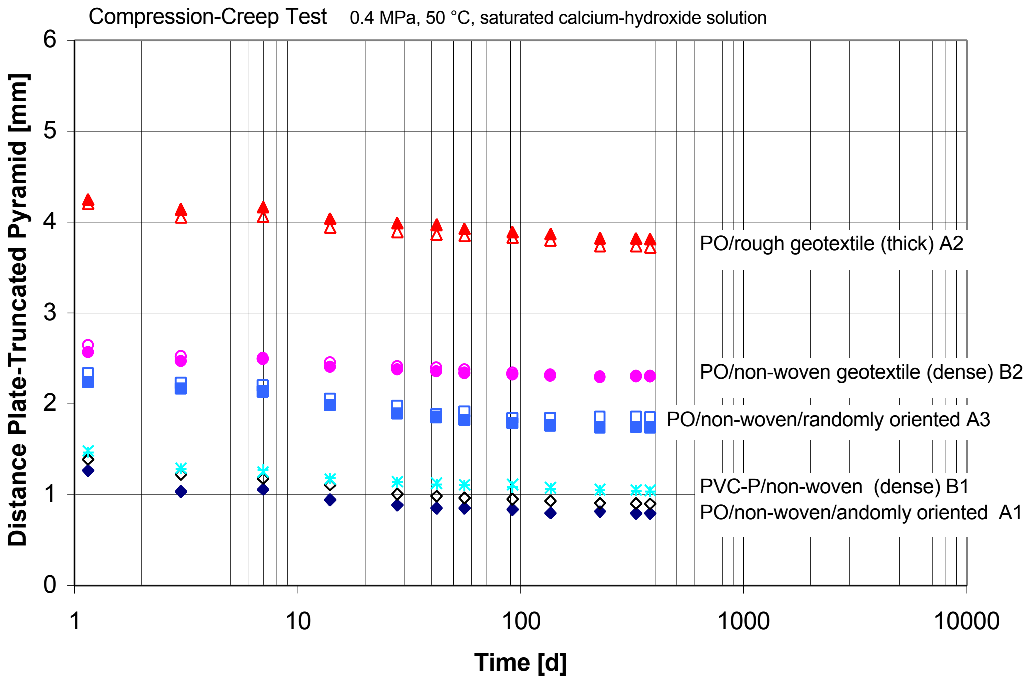


**Diagram 1: Example of deformation curves from compression creep tests for typical WPS at 0.4 MPa distributed load on the ripple plate in alcaline medium at 50° C.**

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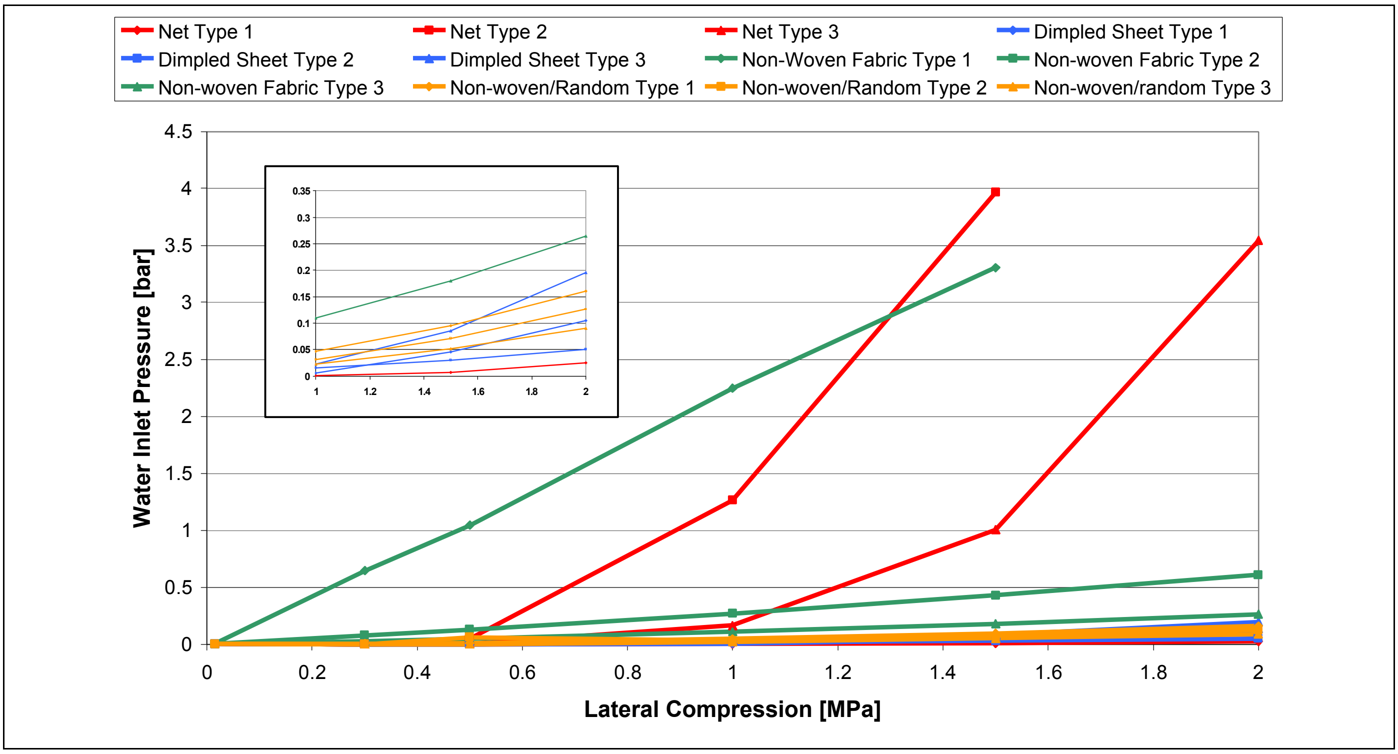


**Diagram 2: Short-term compression/shear test: Inlet pressure of the drainage water (drainage throughput of 10 l/min) versus lateral compression load for the most important types of WPS at 45° C, surface roughness 3.1 mm, the insert shows details of the bottom right hand part.**

### 4.3 Compression/Shear Behaviour

The most severe criterion prooved to be the drainage capability under lateral compression at elevated temperature (+45° C). This capability directly correlates with the time-dependent remaining thickness of the WPS. Several WPS did not meet the requirement of a maximum water inlet pressure of 0.25 bar at 10 l/min throughput in both the short- and long-term (7 days) tests (Diagram 2). The compression/shear results of the limited number of materials tested seem to indicate the following pattern for the behavior of the WPS. All WPS with dimpled sheets prooved fairly well suited in spite of large deformations. Depending on the hardness, several net-type drainage materials intruded into the waterproofing membranes, starting at lateral compression loads of 1 MPa, thus limiting the water flow and finally interrupting it at loads above 1.5 MPa. Some WPS with net-type drainage materials of a modified shape and combined with a less compliant waterproofing membrane, on the other hand, fulfilled the criterion at the highest lateral compression load of 2 MPa in the short-term, as well as in the long-term test (7 days) at loads of 1.5 MPa. WPS with thick and dense non-woven fabrics as drainage materials showed an increase in water inlet pressure beyond the limit even at lateral compression loads of 0.5 MPa. Some waterproofing membranes were perforated after shear displacements of 10 mm under relatively low compressive loads (0.3 MPa), while displacements of 3 mm under the same load did not yield perforations. Passing the compression/shear tests may thus crucially depend on the specific combination of drainage material (structure and shape) and waterproofing membrane (hardness and thickness).

**Figure 8: View of the partially exposed waterproofing membrane in the experimental gallery, the concrete support shell lies on the bottom**

**Figure 9: Vertical folds of the PVC-P waterproofing membrane, extending up to 5 cm into the concrete**

### 4.4 Suitability for Installation in the Experimental Gallery

Before the product tests were effected, three trial tests with typical WPS made from plasticized PVC and PO-Copolymers (dimpled sheet- and net-type drainage materials) yielded the test parameters. The systems were installed on shotcrete surfaces with a variable surface roughness between 3 to 7 mm and a waviness of 4:1, 7:1 and 15:1. With additional heating during the setting of the concrete the temperature in the waterproofing system rose to 55° C. After completion of the construction the drainage capability was determined. The concrete support shell was then removed and the waterproofing system exposed (Fig. 8). These tests yielded unexpected, interesting results. Both types of WPS showed deep folds in the waterproofing membrane, protruding into the concrete after the support shell had been removed (Figs 9, 10). The folds mostly ran along the vertical and extended for about 5 cm with radii of curvature of up to 3 mm. Practically, the folds are narrowly localised but highly strained zones that potentially can function as water drains. There, the waterproofing membrane is strained beyond its apparent yield point and oriented, respectively and is stressed both chemically and physically by the mountain water. The third trial test prooved that WPS can be installed without fold formation. The waterproofing membrane was not damaged in any of the tests. The large waviness of the shotcrete surface, however, required additional expenditures during installation, in particular for fixation and joints. Based on the understanding gained from these tests, the requirements for the main tests could be defined and the test set-up be adapted.

**Figure 10: Back-side of the concrete support shell after its removal, up to 5 cm deep grooves in the concrete.**

## 5 KNOWLEDGE GAINED

It took about 12 months until equilibrium states and recognisable ageing trends were established for the different materials. After 24 months, the loads/exposures yielded clear signs of deterioration and only a few WPS and WPS-components, respectively, fulfilled all requirements. While the waterproofing membranes, on one hand, failed under some specific load conditions, the drainage materials with their formulation, on the other hand, prooved to be rather unsuited for use in hot, aqeous media. The WPS selected for submission by the applicants were mostly manufactured using formulations developed for standard construction applications instead of being designed for the special requirements of the tunnel constructions. The standards usually do not require test durations beyond about 8 months, usually even shorter, and leave the definition of requirements to those responsible for the construction.

Development of folds in the plane of the waterproofing membrane produces potential failure lines and have thus to be avoided in long-term service. This requirement represents a major challenge for the usual installation procedures. The causes for folding have to be thoroughly investigated. Waviness of the backplane, the number of fixation points, the joining procedure, friction between waterproofing membrane and drainage material, the form-boards of the inner concrete shell and the temperature raise caused by the setting of the concrete are among the factors that cause surplus membrane material to protrude into the concrete at the time of construction. Preliminary experiments have shown possible ways of avoiding fold formation.

The evaluation procedure has prooven to be both practical and selective. Compared with established standard test criteria, the requirements have been increased and the evaluation takes considerably longer. In order to design for ageing resistance during the intended service life, test durations of up to five years would be necessary under current test conditions. With the introduction of the evaluation procedure, the behaviour of both, waterproofing membrane and drainage material, acting together as a system, has been investigated under very high load and long-term exposure for the first time.

## 6 PRESENT STATE OF THE EVALUATION

By continuously informing about intermediate results and based on reports summarizing the first 6 and 12 months, the builders could continually assess the situation and adapt it to the construction program. Based on the results of the first test series a shortened evaluation procedure of 12 months duration was re-started. This program evaluated so-called "improved" waterproofing systems – systems modified within certain limits – in the same way and was recently completed. Before the final decision was taken whether a WPS passed the requirements, the results from the second test series were correlated with those from the first, since the 12 months test duration is too short to allow a definitive assessment.

The improvements essentially comprised an increased compression stiffness of both waterproofing membrane and drainage material and modified formulations for increased resistance. Certain criteria still proved to be problematic, in particular the exposure of drainage materials to oxygen-enriched and to alcaline water. The test program on improved WPS up to present yielded 8 systems that passed all requirements. These combine polyolefine- or PVC-P-based waterproofing membranes with either dimpled sheets or composite drainage materials.

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The evaluation procedure is still open for new waterproofing systems. Due to the stringent requirements, other applications in civil engineering and tunnel construction with long service life-times should also be covered. Nevertheless, the effects on the waterproofing system have to be carefully analysed, assessed, and investigated accordingly.

## 7 ACKNOWLEDGMENTS

The authors would like to thank AlpTransit Gotthard AG and BLS AlpTransit AG, as well as the industrial partners for the collaboration and support and acknowledge the project administration by Mr P. Rietmann (E. Basler+Partner AG).

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# Durability Of Concrete With Recycled Aggregates – Results Of Dutch Laboratory And Pilot Tests

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**Summary**: **Recycled aggregates are applied increasingly in many European countries, Japan, and locally elsewhere around the world, notably in densely populated regions. They are obtained from the processing of stony construction and demolition waste. It is estimated that in Europe alone, at least about 200-300 million tonnes of building and demolition waste is currently land filled or applied in road construction.**

**However, recycled aggregates may also be applied as aggregate in concrete, thereby upgrading its application. Concrete's, containing up to 100% fine and coarse aggregates may be utilised successfully. Significant savings on primary aggregates (in the order of 20-25%) may be obtained, thereby contributing to an improved sustainable raw materials use and exploitation, and providing a first step towards the eventual strive for a "closure" of the concrete life cycle loop.**

**Introduction of "new" raw materials in the building industry is only possible when there is no discussion on aspects such as the product quality and the ratio between price and performance. Therefore, this paper will give attention to results obtained in the mixture proportions of concrete with recycled aggregates, strength, durability as well as environmental performance. It will be shown that for the most common ordinary concrete's durability is not a topic of much concern, based on results of tests on creep behaviour, chloride-ion ingress, sea water resistance and freezing and thawing resistance.**

**Keywords.** **ASR, chloride, creep, recycled aggregates, seawater ingress, shrinkage**

## 1 INTRODUCTION

### 1.1 General background

Recycled aggregates, obtained from the processing of stony construction and demolition waste (CDW) are applied increasingly in many European countries, Japan and locally elsewhere around the world, notably in densely populated regions and Japan. It is estimated that in Europe alone, at least about 200-300 million tonnes of building and demolition waste is currently land filled or applied in road construction (Lauritzen, 1998; Symonds, 1999; Hendriks & Pietersen, 2000). This equals an amount of 0.6-0.9 ton per capita per year and is 2-3 times the amount of domestic waste produced per capita per year.

Traditionally, in most countries, CDW, which may contain low percentages of hazardous substances such as asbestos fibres and polycyclic aromatic substances, has been deposited in normal landfills.

Its stony components are in many cases applied as foundation material for (rural) roads (e.g. CUR, 1995; Hanson, 1989). However, the huge quantities involved are causing ever-increasing spatial and environmental problems, most noteworthy in densely populated countries.

In most European Member States increasing land filling costs and special taxes for the disposal of CDW force the construction industry, demolition contractors and recycling companies to look for opportunities for either re-use, recycling and/or recovery.

### 1.2 Recycled aggregates – future scenario's

Since about two decades, an active policy has been carried out in order to increase the overall quality of (notably stony) CDW in the Netherlands, thus facilitating acceptance and application as (a component within) construction materials.

As a result of this active policy, the total level of recycled CDW in the Netherlands has increased from about 60% in 1990 to above 95% in 1999.

A landfill ban for recyclable CDW, effective since 1997, has stimulated this notably.

Furthermore, the establishment of realistic quality criteria and quality control procedures, technical and environmental standards and practical applications, facilitated by demonstration projects and an active government promotion (Dutch policy on sustainable building).

However, almost 95% of the current applications of CDW (recycled aggregates) are still used in road foundations. The application of recycled aggregates as an alternative for natural aggregates in concrete so far remains limited to demonstration projects. Current scenarios predict a steady decrease in the amount of recycled aggregates, which may be applied as road foundations in the next decades, due to a decrease in new large civil engineering projects, and an increasing competition with other materials of secondary origin. Increasing the application of recycled aggregates from CDW as aggregates in concrete is therefore a serious alternative.

### 1.3 Standardisation and quality control in the Netherlands

The Dutch standard VBT (NEN 5950, 1995) is applicable since 1995 for all concrete up to a characteristic strength of 65 MPa and all relevant environmental classes (equivalent to specific maximum levels of W/C). According to this standard, 20% of the natural aggregate may be replaced by recycled concrete aggregate or recycled mixed aggregates, without a need for additional testing of the concrete. Since 1997 the Dutch standard on aggregates for concrete has been updated to accommodate for the application of specific recycled aggregates, provided their specific mass is over 2000 kg/m$^3$.

Most of these recommendations have been incorporated in the advice of the "CEN TC-154 ad-hoc group on recycled aggregates" (CEN, 1998). In the Netherlands, quality control procedures for recycled aggregates have been prepared and are currently applied on a regular basis by all participants involved. On a European level, a Brite Euram thematic network has been established since 1998 to facilitate information exchange, co-ordinate national research on the use of recycled materials as aggregates in the construction industry, and promote its application (Brite-Euram, 1998).

### 1.4 Current research needs

In the Netherlands, roughly 80% of the annual concrete production of about 16 million m$^3$ is used for ordinary concrete in the strength class up to 25 MPa, (i.e. almost 13 million m$^3$). The demand for stones and gravel (4 - 32 mm) and sand (0 – 4 mm) for this concrete is respectively about 14 million and 9.5 million tonnes. Although the production rate of this concrete far exceeds the production of recycled aggregate, the application of this material as <u>an aggregate into concrete</u> is - as a concept – interesting. It contributes to closing the concrete product life cycle (e.g. 7) and may also solve foreseen future outlet problems (notably in the road construction industry) (Hendriks & Pietersen, 2000).

The current Dutch VBT (1995) standard is generally believed to be too restrictive for the majority of the concrete produced, with respect to the formal 20% replacement level. Results of recent research carried out in the Netherlands (see: Hendriks & Pietersen, 2000) points towards the possibility of a 100% replacement of natural course aggregates by recycled aggregates in concrete up to strength class B25 and environmental class 2 (7, 8), by making use of superplasticisers.

Together with the production of coarse recycled aggregates, also fine recycled aggregates (0-4 mm) are produced: the production ratio of coarse to fine aggregates is approximately 10: 3.5.

These recycled fine aggregates currently suffer from a lack of suitable outlets, mainly due to a market over-supply, and technical compomplications. Research has been carried out to test the suitability of these recycled fine aggregates into concrete, with somewhat ambiguous results (CUR, 1995: Fraaij et al. 1999). Depending on the resource, replacement levels of 25 to 50% could be reached in a B25 concrete without course recycled aggregates (9).

### 1.5 Concrete mix design – the pilot test

The Dutch Ministry of Transport, Public Works, and Water management (Directorate General for Public Works and Water Management, Civil Engineering Division) requested in 1998, based on the developments outlined in the previous section, to investigate the possibility to apply high percentages of both fine as well as course aggregates into concrete.

The concrete to be tested had to be of strength class B35, consistency class of 3 (flowable) as well as environmental class 3 (suitable to withstand a marine environment) according to VBT (1995).

The government intention is to apply a suitable concrete quality with a high level of recycled aggregates as a demonstration project in a marine environment (the potential project itself was originally intended to be carried out in 2000, but the current status is that it is delayed).

The concrete mixtures had to contain aggregate replacement levels as high as possible, and had to be tested on workability, strength, and several durability aspects such as chloride ingress, sea water ingress, relative freezing and thawing resistance, potential susceptibility for alkali silica reactivity and finally shrinkage and creep behaviour.

The recycled aggregates itself had to fulfil the specification of the NEN 5905 (1997). Full details have been reported elsewhere (Fraaij et al, 1999).

## 2 MATERIALS

### 2.1 Cement type

A standard Dutch CEM III 42,5 LH HS ground granulated blast furnace slag cement, with a slag percentage of 72%, was used. This type of cement is regularly applied in the Netherlands for concrete constructions in a marine environment, for several decades.

### 2.2 Aggregates

The natural coarse and fine aggregates consisted of river gravel (4-32 mm) and river sand (0-4 mm) respectively. Both a recycled mixed aggregate and a recycled concrete aggregate (4-32 mm) were used as partial replacement of the natural aggregates. A washed recycled crusher sand (0-4 mm) was used as replacement for the natural sand. All aggregates fulfilled the specifications of NEN 5905. Since 1997, this Dutch standard is applicable for both natural as well as recycled aggregates.

### 2.3 Superplasticiser

A sulphonated naphthalene formaldehyde condensate superplasticiser was used in most mixtures. In some mixtures a polycyclic sulphonate superplasticiser was used instead.

### 2.4 Mixture proportions

Table 1 presents a survey of the mixtures. Due to the high "environmental class" of concrete in contact with sea water, with an additional (though less frequent) freezing and thawing attack, the choice is either to use an air-entrainer or to use a W/C = 0.45. It was decided to prepare mixtures with a W/C of 0.45 without air entraining agent, taking into account the requested characteristic strength of 35 MPa and the specific cement type used.

**Table 1. Concrete mixtures. Data present relative volumes: The total of the fraction < 4 mm is stated as 100% and the total of the fraction 4 - 32 mm is taken as 100% as well.**

| *Mixture* | *Fraction 0 - 4 mm (100% total)* | | *Fraction 4 - 32 mm (100% total)* | | |
|---|---|---|---|---|---|
| | *River sand* | *Breaker sand* | *River gravel* | *Mixed aggregates* | *Concrete aggregate* |
| 1 (reference) | 100 | 0 | 100 | 0 | 0 |
| 2 | 100 | 0 | 0 | 100 | 0 |
| 3 | 100 | 0 | 0 | 0 | 100 |
| 4 | 75 | 25 | 0 | 100 | 0 |
| 5 | 75 | 25 | 0 | 0 | 100 |
| 6 | 50 | 50 | 0 | 100 | 0 |
| 7 | 50 | 50 | 0 | 0 | 100 |

Additional remarks Table 1:

1. Mixed aggregate: mixture of broken masonry debris and concrete debris
2. Concrete aggregate: broken concrete debris
3. Cement: 350 kg/$m^3$ blast furnace slag cement (with 25% portland clinker and mortar strength after 28 days of 45.5 MPa average value)
4. Air content: about 1.5%

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# 3 RESULTS

## 3.1 Workability

The new generation of superplasticisers (high range / high performance super-plasticisers) enabled a reasonable workability up to about one hour after mixing. The recycled materials, and especially the breaker sands, have a negative influence upon the workability. The concrete mixtures under investigation displayed a somewhat sticky behaviour compared to mixtures with a higher W/C but similar workability (a slump value of about 80 mm). The problem with recycled aggregates is the somewhat unpredictable behaviour concerning water suction (when partially dried) or water release (when particles are wet). Also the irregular edginess plays a role in the rheological behaviour.

Some trial mixtures have been made with completely wetted aggregates. However, despite the foreseen water release, the actual W/C (recalculated after the determination of the volumetric and air content in the vibrated fresh mix) became too high. The actual mixtures were therefore made with fully dried aggregates, taking into account that the aggregates will absorb water in the fresh stage of the concrete with a speed indicated by Fig. 1. Most subsequently prepared mixtures displayed an actual W/C of about 0.45, though some mixtures showed an actual W/C smaller than 0.45 while others resulted in higher values than 0.45. Only those concrete mixtures with a recalculated W/C of about 0.45 will be discussed in this paper.

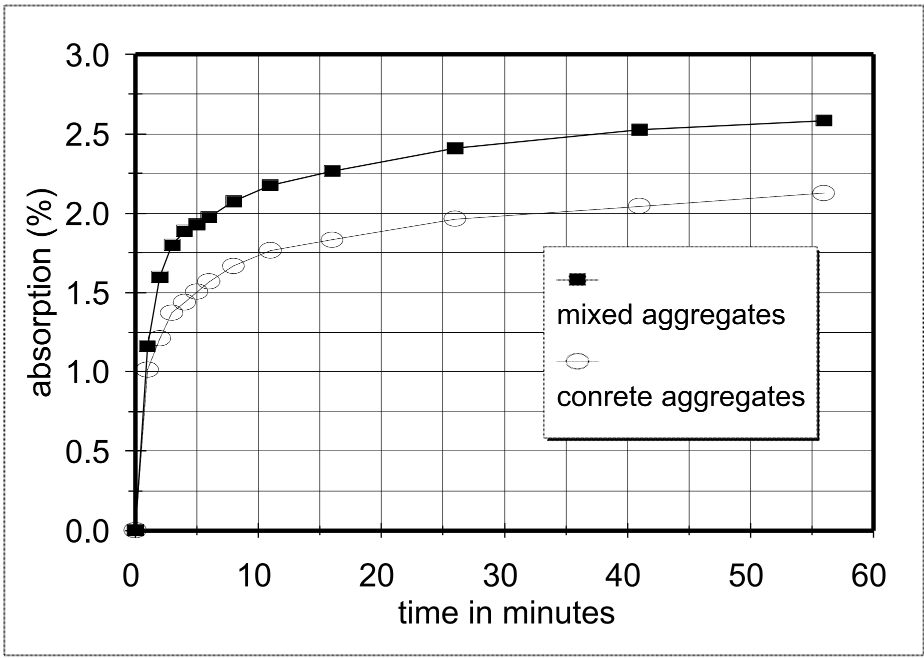


**Fig. 1. Moisture absorption progress in recycled aggregates (start at dried condition).**

## 3.2 Strength

The strength appears to be influenced by the actual W/C in a much more pronounced way than by the type and the amount of recycled material (however in this paper the strength as a function of W/C is not given since this is common knowledge). Table 2 gives a summary of results for mixtures with a W/C of about 0.45. The mixtures with recycled mixed aggregates showed a slightly lower strength than the concrete with recycled concrete aggregates (with about the same W/C). As can be seen from table 2, at higher amounts of recycled sands it is increasingly difficult to obtain a mixture with the desired W/C and with the right workability as well.

## 3.3 Durability

### 3.3.1 Chloride ingress:

The chloride penetration has been measured according to the AEC laboratory standard APM302 (1999) on Concrete testing of chloride penetration (immersion of a specimen during 5 weeks in a NaCl solution of 165 gram/litre, intrusion from one side). Fig. 2 gives results for mixtures where river gravel 4 – 32 mm has been replaced completely by recycled materials. In all cases, the intrusion depth remained limited to about 20 mm. Clearly the mixtures with recycled mixed aggregates showed a somewhat higher superficial intrusion than the mixtures with recycled concrete aggregates.

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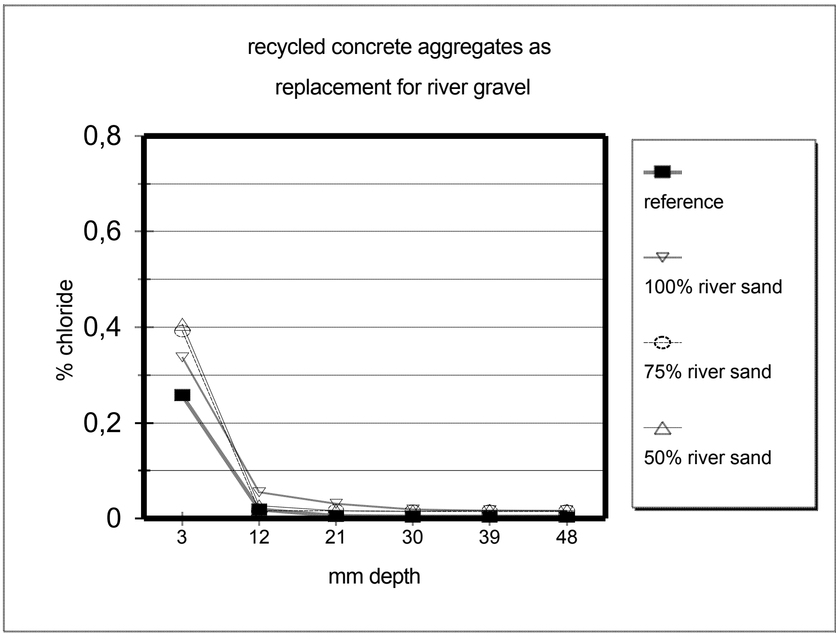


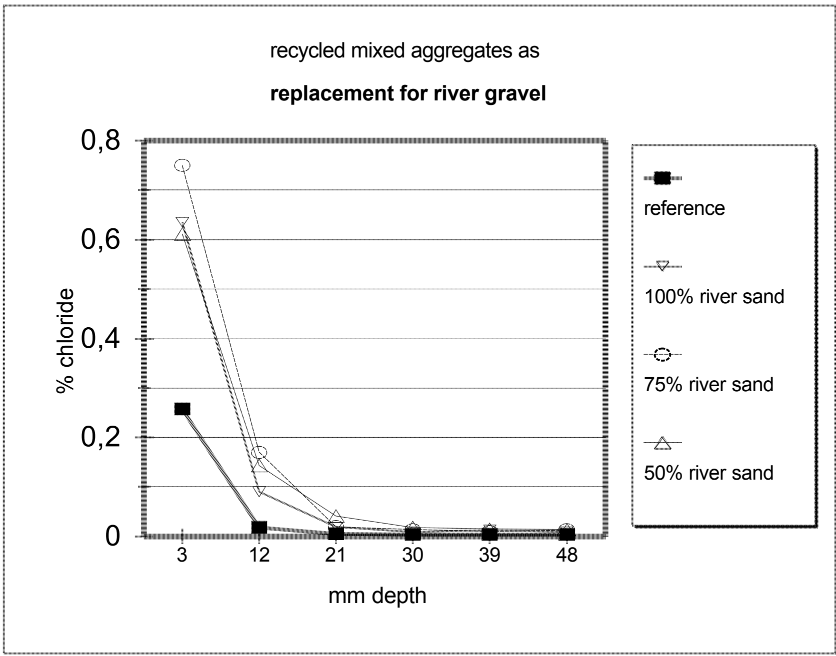


**Fig. 2. Chloride penetration in concrete mixtures with W/C of about 0.45. In case of 75% river sand or 50% river sand in the fraction < 4 mm the amount of recycled material is 25% or 50% respectively (vol/vol).**

**Table 2. Average cube compressive strength in MPa of mixtures with recycled materials and W/C of about 0.45 (except the mixtures with *)).**

| *% v/v breaker sand 1) ?* | *% (vol/vol)* ***recycled mixed aggregate*** *at the expense of river gravel fraction 4 - 32 mm. The total of the fraction 4 - 32 mm (river gravel + mixed aggregate) is put at 100%* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *0* | | *50* | | *80* | | *100* | |
| | *1 week* | *1 month* | *1 week* | *1 month* | *1 week* | *1 month* | *1 week* | *1 month* |
| 0 | 37.7 | 53.7 | 41.0 | 45.4 | 37.9 | 45.7 | 35.6 | 44.4 |
| 25 | 42.2 | 55.4 | | | | | 34.8 | 42.7 |
| 50 | 35.6*) | 44.0*) | 37.3 | 44.9 | 36.2 | 43.1 | 32.9 | 42.1 |
| 100 | 41.6*) 2) | 54.6*) 2) | | | | | 33.1 | 41.1 |
| *% v/v breaker sand 1)* | *% (vol/vol)* ***recycled concrete aggregate*** *at the expense of river gravel fraction 4 - 32 mm. The total of the fraction 4 - 32 mm (river gravel + mixed aggregate) is put at 100%* | | | | | | | |
| 0 | 37.7 | 53.7 | 42.6 | 48.7 | 40.7 *) | 50.5**) | 41.1 | 47.2 |
| 25 | 42.2 | 55.4 | | | | | 38.9 | 50.8 |
| 50 | 35.6*) | 44.0*) | 44.9 | 57.6 | 42.7 | 50.7 | 40.9 | 50.1 |
| 100 | 41.6*) 2) | 54.6*) 2) | | | | | 27.6*) | 34.3*) |

*) W/C > 0.46

**) W/C < 0.44

1) total fraction river sand + recycled breaker sand (0 - 4 mm) is put at 100% (vol/vol)

2) Unpredictable high strength; presumably W/C is actually smaller than 0.45.

3.3.2 Sea water swelling:

Concrete specimens (75x75x150 mm) were sawn out of hardened cubes 150x150x150 mm (4 per cube) and were put in artificial seawater with a threefold salt concentration compared to normal seawater. Concrete with recycled mixed aggregates displayed an increased swelling compared to concrete with recycled concrete aggregates, as can be seen in Figure 3. However, there was no evidence for cracks due to sulphate swelling. It appears that the swelling is partially caused by the water absorption capacity of the porous recycled aggregates.

3.3.3 Freezing and thawing:

The resistance against freezing and thawing has been tested according to the Fagerlund method in which samples were wetted to a certain moisture content (expressed as % saturation compared to fully saturated specimen) and exposed to freezing and thawing cycles. The dynamic Young's modulus is calculated from the pulse time velocity of a sound pulse before and after 12 freezing and thawing cycles. First damage $F_d$ is defined as the % moisture saturation where the Young's modulus starts to decrease. The critical saturation degree $S_{crit}$ is defined as the % moisture saturation at the 90% decrease level of the Young's modulus due to freezing and thawing cycles. Table 3 gives results for the concrete mixtures with W/C of about 0.45. Mixtures with recycled mixed aggregates are somewhat more prone to freezing and thawing degradation than mixtures with recycled concrete aggregates.

3.3.4 Alkali silica reaction:

Alkali silica reaction has been investigated according to the standard NEN 5925 that prescribes the production of standard mortar bars with the particular aggregate to be tested on reactivity. The procedure involves the length change of the bars under certain conditions. Unfortunately the recycled aggregates gave unworkable mortars due to the water absorption of the porous material. Therefore it was necessary to use superplasticisers to provide for a reasonable workability. None of the recycled aggregates caused cracking or substantial length changes during the test. This was confirmed by PFM.

**Table 3. Critical degree of saturation $S_{crit}$ and saturation at first damage $F_d$ after 12 freezing and thawing cycles.**

| *Recycled coarse aggregate 4 – 32 mm (all river gravel replaced) ?* | *% (Vol/vol) breaker sand as replacement for river sand. Total fraction < 4 mm = 100%* | | | | | |
|---|---|---|---|---|---|---|
| | *0%* | *25%* | *50%* | *0%* | *25%* | *50%* |
| | *Critical degree of saturation $S_{crit}$* | | | *First damage % saturation $F_d$* | | |
| Recycled concrete aggregate | 92 | 89 | 90 | 90 | 84 | 87 |
| Recycled mixed aggregate | 91 | 89 | 78 | 70 | 83 | 65 |
| Reference concrete with river sand and river gravel | 100 | | | 82 | | |

3.3.5 Shrinkage and Creep:

The shrinkage is influenced by the porosity of the recycled materials. Concrete with recycled mixed aggregate (4–32 mm) shows a smaller shrinkage than with recycled concrete aggregate. Indeed, the recycled mixed aggregate is more porous than the recycled concrete aggregate and the river gravel thus provides a water buffer against drying out, (Figures 4 and 5).

The creep of concrete with recycled mixed aggregates is higher than with recycled concrete aggregate. Figures 6 and 7 give the creep function (for a loading of 1 MPa) and are corrected for the shrinkage effects. The actual loading was 20% of the compressive strength of the concrete.

## 4 CONCLUSIONS AND RECOMMENDATIONS

Is it well possible to produce moderate strength (35 MPa) and environmental class 3 concretes (for application in a marine environment) by replacement of relatively large amounts of primary aggregates by secondary raw materials, tested according to NEN 5905. This study indicates that especially recycled concrete aggregate (fulfilling the Dutch NEN 5950) may be applied successfully, without potentially disturbing effects. However, notably the porous (fraction in) mixed recycled aggregates are suspected to be responsible for the sometimes erratic - and consequently less reproducible - W/C, resulting in ad-hoc occurring workability differences.

In this study, the amount of water that has been added to the mixture has deliberately been suppressed. This was done to ensure that the actual W/C (after mixing, casting and vibrating) did not exceed 0.45. The problem involved by the choice of this approach was a decreased workability, compared to mixtures with lower replacement levels of recycled aggregates. Only the modern generation of high range superplasticisers seemed to be able to maintain a correct workability within a realistic time span.

The following conclusions and recommendations can be made:

1. It is necessary to make use of the modern generation of high range superplasticisers for the concrete compositions studied;
2. The actual W/C that is found in the fresh mixtures after casting and vibrating is very difficult to predict. Therefore one should remain on the safe side and make mixtures that are actually “dryer” than requested;
3. It is possible to make concrete with recycled aggregates in which the total fraction 4 – 32 mm of the natural aggregate is totally replaced by recycled aggregates. It is also possible to replace a part of the fraction < 4 mm by recycled breaker sand, but it is not recommended to replace all the river sand by recycled breaker sand; a maximum replacement level of 50% is acceptable in practice, mainly for reasons of workability;
4. There is no evidence for potential alkali silica reaction with the types of recycled aggregates that were used in this study, tested according to NEN 5925 and PFM (method now under revision);
5. Concrete containing recycled mixed aggregates all displayed a somewhat increased seawater swelling, creep and a higher chloride intrusion, compared to concrete containing recycled concrete aggregate. Reference concretes with river gravel and the river sand displayed the lowest values;
6. The sensitivity for freezing and thawing attack increased slowly with the amount of material that is replaced by recycled aggregates; and
7. Further research to provide a more detailed understanding of the causes for the comparably high variations in porosity (or specific mass) of recycled aggregates is needed.

## 5 ACKNOWLEDGEMENTS

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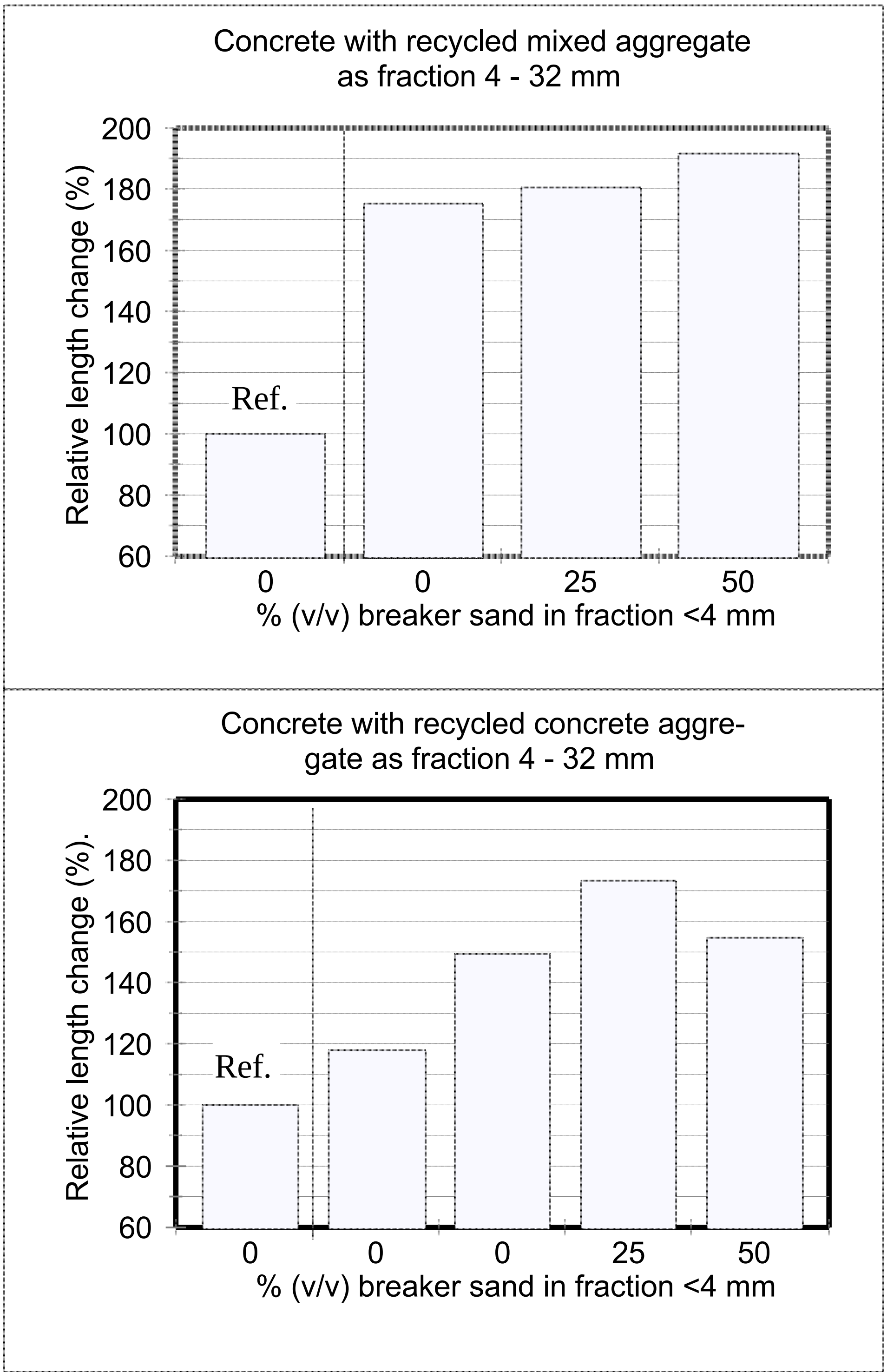


**Fig. 3. Swelling after 6 months in artificial seawater (with triple seawater salt concentration) relative to swelling of reference concrete. Swelling of reference concrete with river sand and river gravel is put at 100%. All mixtures were prepared with a W/C of about 0.45. Results have been averaged from 4 samples per mixture. The graph for recycled concrete aggregate shows 2 columns in case of 100% river sand due to two different mixtures.**

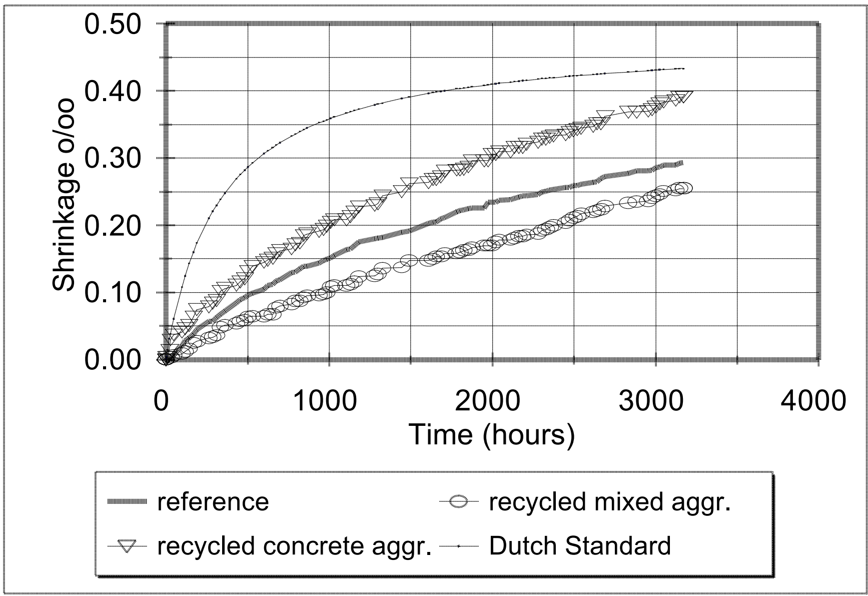


**Fig. 4. Shrinkage of concrete where half of the sand fraction (< 4 mm) has been replaced by recycled breaker sand. Gravel fraction (4 – 32 mm) is either recycled mixed aggregate or recycled concrete aggregate. Reference concrete is with river sand and river gravel. Concrete with W/C of about 0.45.**

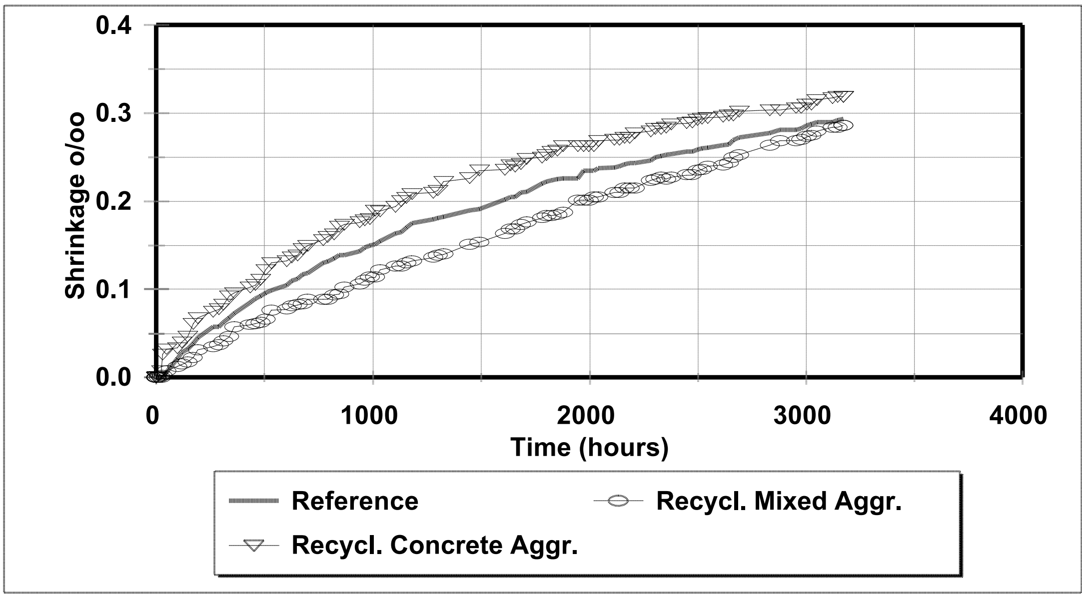


**Fig. 5. Shrinkage of concrete with river sand as sand fraction (< 4 mm). Gravel fraction (4 – 32 mm) is either recycled mixed aggregate or recycled concrete aggregate. Reference concrete is with river sand and river gravel. Concrete with W/C of about 0.45.**

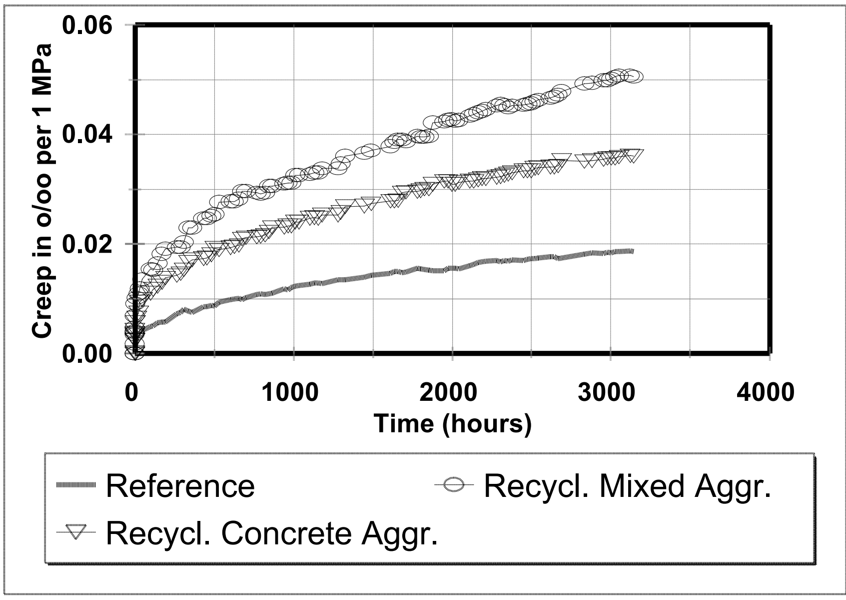


**Fig. 6. Creep function of concrete where half of the sand fraction (< 4 mm) has been replaced by recycled breaker sand. Gravel fraction (4 – 32 mm) is either recycled mixed aggregate or recycled concrete aggregate. Reference concrete is with river sand and river gravel. Concrete with W/C of about 0.45.**

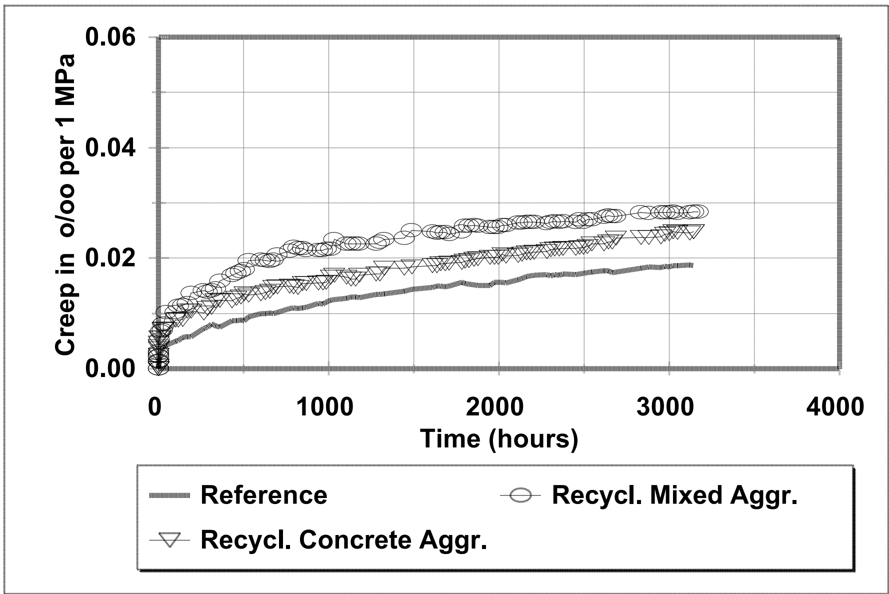


**Fig. 7. Creep function of concrete with river sand as sand fraction (< 4 mm). Gravel fraction (4 – 32 mm) is either recycled mixed aggregate or recycled concrete aggregate. Reference concrete is with river sand and river gravel. Concrete with W/C of about 0.45.**

# Durability And Life Cycle Aspects On Bio-Fibre Composite Materials

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**Summary: With the appearance of synthetic materials at the beginning of the last century, synthetic based materials have steadily replaced bio-based products. As a result of this change in raw material utilization, combined with an increase in energy and chemical demand, the world is now facing an ecological crisis. This crisis will greatly intensify with the expected growth in demand for industrial products in developing countries. Thus the world community is facing a challenge in having to decrease pollution levels, while at the same time significantly increasing industrial output. Such predictions have led to support for enhanced industrial use of renewable resources (bio-fibre) at the expense of non-renewable resources (fibre-glass). Bio fibres may therefore face a renaissance, not only for old industrial products but also for the manufacture of new types of materials and products. Industrial use of bio fibres as woodfibres for the manufacture of pulp, paper, fibreboards and particleboards are well established in the world. The renewed interest in industrial use of bio-fibre composite materials within the automotive, building, furniture and packaging industries has led to development and production of many bio-fibre based composite products. Several companies in the building material industry worldwide are interested to produce products based on bio-fibre composites. It can be products for outdoor decking as roof plates, wall boards, windows and door panels. Most of the fibres that are used for these composites are from short woodfibres but some parts come from long jute-, flax- and hempfibres. One main topic for such composite materials concern the durability and the life cycle assessment. Based on literature studies this paper will present and discuss durability and life cycle aspects on bio-fibre composite materials for the building industry and other industries. Special focus will be on products, which are exposed to the climate and placed in the outside scale of the house such as roof plates, facade materials, doors and windows.**

**Keywords: Durability, biofibre composite materials, roof plates, facades, doors.**

## 1 INTRODUCTION

The world community is facing a challenge in having to decrease pollution levels, while at the same time significantly increasing industrial output. Such predictions have led to a need for enhanced industrial use of renewable resources (bio-fibres) at the price of non-renewable resources (fibre-glass). Bio fibres will therefore face a renaissance, not only for old industrial products but also for the manufacture of new types of materials and products.

The renewed interest in industrial use of bio fibres within automotive, building, furniture and packaging industries of European countries has led to development and production of many bio-fibre based products. A number of biobased products are made from *mats*, which are made of bio fibres or a mixture of bio fibres, plastic fibres and resins in varying amounts. Mats may be used in a number of products, e.g. filters for air and water, growth media for production of vegetables, flowers and grass mats. Bio-fibre geotextiles are already available on the market. Bio-fibre mats also have a promising future as insulation material within the building industry. The market potential for partial replacement of glass and mineral wool mats for insulation is huge. *Composites* for structural and non-structural purposes are manufactured by hot pressing of mats with success within the car industry. Fibre/polymer and fibre/cement composites can be manufactured through extrusion or injection moulding. New composites can be designed to have the required stiffness and strength. The importance of the role of final product design can not be overstated.

The annual global production of bio fibres is today about 4 billion tons of which roughly 60 % come from agricultural crops and 40 % from forests. In comparison, annual world production of steel is currently around 0,7 billion tons and plastic production is about 0,1 billion tons. As an example Sweden´s annual production of bio fibres is about 50 million tons from forestry and about 0,1 million tons from agriculture. Taking the above figures in consideration, it is apparent that there should be more than sufficient volumes of agricultural fibres available globally for new products. The huge production of wood fibres in Sweden is mostly used for pulp and paper products. There is not a great supply of wood fibres for the manufacture of new types of materials and products. Therefore there is a potential of using agricultural fibres and the cultivation of agricultural fibres has to increase to supply the industry with bio-fibres of sufficient quality.

The market for bio fibres in the building industry is closely linked to the ecological building sector. In contrast to conventional construction, ecological building regards the building as an integral part of the whole human and natural environment. Ecological building is a growing sector. As an example this sector acounted for 6% of the building market in Germany in 1995. Surveys show that 42% of private home builders in Germany are prepared to pay more for ecological building materials.

The use of biofibrous materials in the automotive industry is now widespread. Environmental protection, weight reduction and improved recycling characteristics are arguments, which car manufacture frequently advance to familiarize their customers with the ecological benefits of such materials.

An important step towards higher performance applications was achieved with the door panels of the Mercedes-Benz E-Class. These door panels were manufactured by a bio-fibre reinforced material consisting of a flax/hemp fibre mat embedded in an epoxy resin matrix. A weight reduction of about 20 % compared to doorpanels made of fibre glass was achieved and the mechanical properties were improved.

## 2 BIO-FIBRE COMPOSITE MATERIALS

### 2.1 Bio-fibre types and production

Bio fibres are subdivided based on their origins. They are coming from leafs (abaca, sisal, hennequen), seed-hairs (coir, cotton), basts (flax, hemp, jute, ramie) and wood. Most of the bio fibres, which are interesting for Europe are coming from flax and industrial hemp. These varieties are cultivated for both their fibres and their oil. Both flax and indusrial hemp are annual plants. Flax grows up to 1 m high and produce about 1 ton bastfibre per ha. Industrial hemp may grow up to 4 m high and produce about 3-4 ton fibre per ha.

The most important flax growing areas in the world are the former Soviet Union and the EU countries. These countries have about 40 % each of the 500 000 ha flax grown in the world.

During the latest decades the trend has been a long term decreas of acreage both for flax and hemp. However, during the latest years there might have occured a shift of trends with a slightly increasing area due to high interest in Canada and in the EU. Over the last 30 years France has had a rather constant hemp cultivation area of about 10 000 ha. During the 1990´s an area of about 17 000 ha hemp was cultivated per year in China. Totally about 50 000 ha is grown per year for hempfibre around the world.

In table 1 the worldwide production of plant fibres in 1999 divided into fibres and main country producer is shown. This plant fibre production amount is focused on the main fibre plants and does not include straw and wood fibre production.

**Table 1. Worldwide production of Plant Fibres in 1999 (Karus. 2000).**

| Fibre | Main country producer | Production in million tons | Production in % of total |
|---|---|---|---|
| Cotton | China, USA, India, Pakistan | 18.20 | 78.5 |
| Jute | India, China, Bangladesh | 3.37 | 14.5 |
| Flax | EU, China, Eastern Europe | 0.63 | 2.7 |
| Sisal | Brazil, China, Mexico, Kenya | 0.38 | 1.6 |
| Cocos | Indien, Sri Lanka | 0,27 | 1.2 |
| Ramie/Abaca | China, Ecuador, Philippines | 0.27 | 1.2 |
| Hemp | China, EU, Eastern Europe | 0.07 | 0.3 |
| Total | | 23.19 | 100 |

## 2.2 Bio-fibre structure and properties

All plant species are made of cells. When a cell is very long in relation to its width it is called a fibre. The fibre is like a microscopic tube and it consists of cellulose, hemicellulose and lignin. Cellulose is the basic structural component of all plant fibres. The cellulose molecules consist of glucose units linked together in long chains. Hemicellulose are also found in all plants. Hemicelluoses are polysaccharides bonded together in branching chains. Lignin is the compound, which gives rigidity to the plant. Lignin is a three-dimensional polymer with an amorphus structure (Olesen & Plackett, 1999). Pectin is the glue, that holds the fibre bundles together. In table 2 the chemical composition of some selected agricultural and wood fibres is shown.

**Table 2. Chemical composition of some selected agricultural and wood fibres (weight-%) (Olesen & Plackett, 1999).**

| Fibre | Cellulose | Hemi-Cellulose | Lignin | Pectin |
|---|---|---|---|---|
| Jute | 70-75 | 12-15 | 10-15 | 1 |
| Flax,hemp | 68-85 | 10-17 | 3-5 | 5-10 |
| Seed hair, cotton | 89-99 | 3-6 | - | - |
| Wood | 38-45 | 24-39 | 22-34 | 0.5 |

The variation of plant fibre dimensions is great, which is shown in table 3.

**Table 3. Dimensions of some selected plant fibres (Olesen & Plackett, 1999).**

| Fibre | Fibre length (mm) | Fibre width (mm) |
|---|---|---|
| Jute | 2 (1.5-5) | 0.02 |
| Flax | 30 (5-60) | 0,02 |
| Hemp | 20 (5-55) | 0.025 |

The combined effect of the three main constituents results in properties, which are unique for plant fibres. The most important properties are:

- High tensile strength and low density. In relation to its weight the best bast fibre attain strength properties similar to that of fibre-glass.
- High heat, sound and electrical insulating ability
- Biodegrability

The good specific mechanical properties are due to their relative high strength and low density. In table 4 mechanical properties of plant fibres are compared with conventional reinforcing fibres.

**Table 4. Mechanical properties of plant fibres as compared to conventional reinforcing fibres (Biedzki & Gassan, 1999).**

| Fibre | Density (g/cm3) | Elongation (%) | Tensile strength(MPa) | Young´s modulus(GPa) |
|---|---|---|---|---|
| Jute | 1.4 | 1.5-1.8 | 200-450 | 20-55 |
| Flax | 1.5 | 2.7-3.2 | 500-900 | 50-70 |
| Hemp | 1.5 | 1.6-3.5 | 310-750 | 30-60 |
| Sisal | 1.5 | 2.0-2.5 | 80-840 | 9-22 |
| E-glass | 2.5 | 2.5 | 2500-3500 | 70-73 |
| Aramid | 1.4 | 3.3-3.7 | 3500-4000 | 85-135 |
| Carbon | 1.4 | 1.4-1.8 | 2500-6000 | 220-700 |

## 2.3 Bio-fibre composite products

Plant fibres have found application as reinforcement in many cement based or polymer based composite products.

The reinforcement of cement and concrete based products with plant fibres present challenging and exiting construction materials. The major role of the fibres is in delaying and controlling the tensile cracking of the matrix. This function reduces deformations at all stress levels. If properly carried out plant fibre concrete has all the the potentially favourable qualities of normal concrete: fire resistance, is water tight, easy to produce and is inexpensive. The plant fibres can be utilized both in semi-structural elements such as thin sheets and cladding panels as well as in load bearing elements. Other examples of cement based products reinforced with long plant fibres are roof plates and floor slabs.

Three-dimensional composite products for structural and non-structural purposes can be manufactured by hot-pressing of mats. The mats may be made entirely of plant fibres or a mixture of plant fibres, plastic fibres and resins. In addition to hot-pressing of mats bio-fibre reinforced composites can be manufactured by extrusion or injection moulding. Other processing options are thermoforming, vacuum forming and resin transfer moulding. The most used thermoplastics are polypropylene, polystyrene, vinyls and high and low density polyethylenes. The most usable building materials made of plant fibre reinforced plastics are window frames, door panels and wall panels. There are also many applications for plant fibre composites in the automotive, the furniture and the packaging industries.

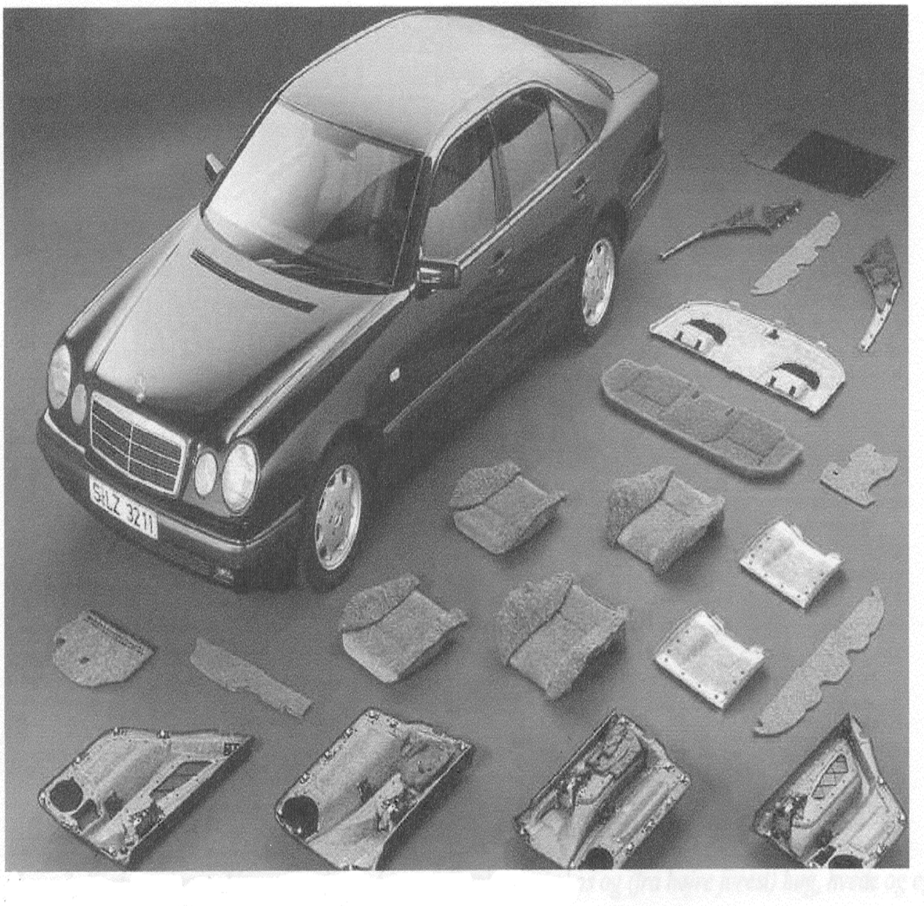


**Figure 1. The use of plant fibres in automotive products of Mercedes E-class (Olesen, 2001).**

## 3 DURABILITY OF BIO-FIBRE COMPOSITE MATERIALS

One of the disadvantages of bio fibres is the prone to water absorbtion. Most of the plant fibres are hygroscopic and the water absorbtion may be rather great. As a consequence of the hygroscopicity products and materials based on plant fibres are not dimensionally stable under changing moisture conditions. The unfavourable absorbtion of moisture by plant fibres is due to the cohesion of hydrogen by hydrogen molecules to groups of hydroxyl in cellulose in cell-wall polymers. Reactive organic chemicals are being tied to the cell-wall hydroxyl groups of cellulose, hemicellulose and lignin.

This absorbtion disadvantage may be controlled at an extra cost by different methods of impregnation e.g. heat treatments or chemical modification procedures as acetylation. Acetylation means introduction of acetyl groups into organic combinations. By replacing some of the hydroxyl groups on the cell wall polymers with bonded chemical groups, the hygroscopicity of the bio-fibre material is reduced. With acetic acid, acetylation reaches probably no other than the easily hydroxyl groups (Biedzki & Gassan, 1999).

Another disadvantage also concerning the durability of bio-fibre composites is the destructive effect of the alkaline pore solution on plant fibres. This effect is well known and established. The net effect of these chemical degradation processes is that with time the fibres will cease to exist and provide no reinforcing property. In the short term the fibres provide cohesion and workability to the cement matrix. In the long term prolonged reinforcing action between the matrix and the fibre can only be ensured by carefully designed procedures. There are also many reports indicating that the lignin content of the fibres may influence the hydration of the cement composite (Swamy, 1990).

### 3.1 Durability of cement based composite products

Roofing is one of the most important and difficult problem in housing. In most developing countries and in many industrialized countries natural materials are used for roofing in rural areas. An interesting solution to the roofing problem seems to be thin sheets made of plant fibre concrete.

When mixing plant fibres in the mortar the fresh concrete may be shaped in a simple manner, which is one advantage with this material technology. Another advantage is that the fibres increase the toughness and impact resistance of the hardened concrete. This means that the material may be used in structural applications. A corrugated roof sheet is probably one of the most interesting forms even if other forms can be produced. Apart from roofing materials plant fibre concrete can be used for production of sun screens, thin-walled blocks, small beams, and other products.

In the 1980´s a joint venture project between research institutes in Sweden and Tanzania was carried out with the aim to investigate observed embrittlement of sisal fibre concrete for roofing sheets and find suitable countermeasures. The research found that the embrittlement of sisal fibre concrete is due to the fact that the alkaline pore water in the concrete reacts with cellulose components in the fibre so that its loadbearing capacity is reduced. It seems that the lignin and the hemicellulose in the middle lamella are decomposed first thus weakening the link between the individual fibre cells. Sisal fibres in alkaline solutions with a pH-value higher than 12 are discoulered yellow and lose a considerable proportion of strength in the wet state. The decomposition of the fibres takes place more rapidly at high temperatures (Gram, et al 1984)

The decomposition process already starts when the sisal fibre concrete is manufactured. The alkaline pore water in the vicinity of the fibre will probably be neutralized by the reactions, which take place with the fibre components. For the embrittlement of the concrete composite the external environment, where the composite is located play a significant role. External changes of moisture and temperature in the environment will influence the transportation of new alkaline pore water to reach the plant fibres and also influence the removal of decomposition products (Gram, 1983).

By replacing about 45 % of the cement with silica fume the alkalinity of the pore water is reduced and the embrittlement of the plant fibre concrete can be decreased. Changing Portland cement to Slag cement or high alumina cement are other countermeasures to reduce the alkalinity but these measures are not sufficent to prevent embrittlement in the composite.

An interesting measure to improve the resistance of the plant fibre composite against embrittlement is to precarbonate the entire composite product. By treating the manufactured composite with carbonic acid or water containing carbonate the carbonation will start and the pH-value of the pore water will decrease. If this method will be successful the fibre reinforcement must be placed in the outer layer of the composite (Gram, et al,1984).

Brazilian research on durability of coir concrete showed that it is possible to increase durability of bio fibres by changing the concrete matrix composition in order to obtain low alkaline media. The research proved that it is also possible to produce load-bearing structures with bio fibres reinforced materials (John, et al. 1990).

### 3.2 Durability of polymer based composite products

Plant fibre reinforced thermoplastic composites can be used for many products within the automotive and the building industries. Research has shown that such products have advantages in terms of cost, weight and environmental factors when compared to similar combinations based on fibre-glass reinforcement.

Current applications for plant fibre reinforced thermoplastic composites in the automotive industry are door panels, interior trim, load floors, package trays and rear shelves. Examples on products in development are door trim, headliners, instrument panels and seat backs.

Plant fibre reinforced thermoplastic composites for the building industry may be products with climatic protective functions and less load-bearing capacity. Examples are window frames, door panels and wall panels. Window frames made of plant fibre reinforced thermoplastic should be competible to corresponding product reinforced with fibre-glass.

The durability of these plastic based composites is mostly depending on the moisture influence.

When manufacturing the plant fibre reinforced composites it is important to dry the fibres carefully before processing because water on the fibre surface acts like a separating agent in the fibre matrix. Another phenomena is that voids may appear in the matrix because of evaporation of water during the reaction process. This is due to the fact that the processing temperarure mostly lies over the evaporation temperature of water. Both these phenomena lead to a decrease of mechanical properties of plant fibre reinforced composites (Biedzki & Gassan, 1999).

By using silanes as coupling agent in jute reinforced polyester resin composites it has been shown that the water repellance was improved and the strength of the composite was increased. This is shown in Fig 2. The improved moisture resistance caused by the application of the coupling agent can be explained by an improved fibre-matrix adhesion. The coupling agent build chemical and hydrogen bonds, which reduces the moisture (Biedzki & Gassan, 1999).

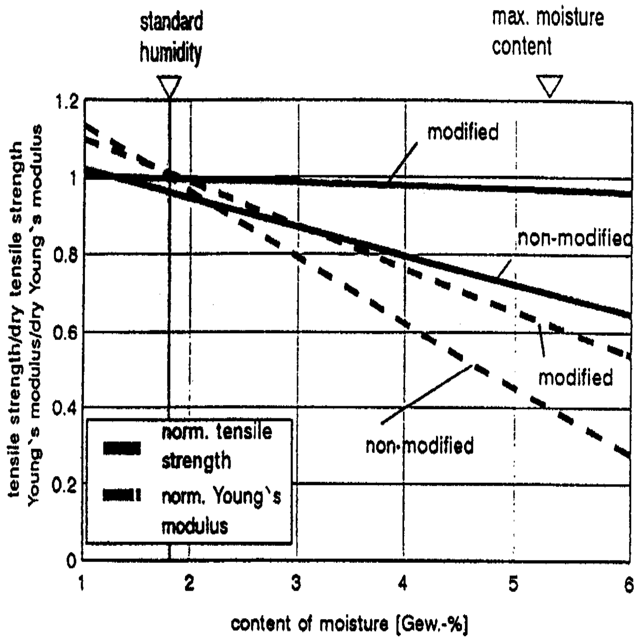


**Figure 2. Influence of silane coupling agents on the strength of jute reinforced epoxy-resin composites at different moisture contents (Biedzki & Gassan, 1999).**

Particle boards and flake boards have been tested for biological resistance to several different types of organisms. Acetylated types of these boards have been tested against termite attack during a two-week test. The tests showed good but not complete resistance against termite attack. Since termites can live on acetic acid and decompose cellulose to mainly acetic acid it is not surprising that the resistance is not completely (Rowell et al., 1997).

Acetylation has been shown to improve ultraviolet resistance of aspen fibre board. Accelerated weathering tests of acetylated aspen fibre boards show reduced weight loss and penetration depth. Acetylation reduces surface erosion by 50 % and the lignin content is also greatly reduced in the surface as a result of the weathering. Because of the acetylation cellulose and hemicellulose are much more stable to photochemical degradation (Rowell et al., 1997).

## 4 DISCUSSION

Today we see a fast development in the industrialized world, primarily in USA and Europe, towards utilization of biofibres as an industrial raw material. This change from synthetic oil based plastics towards environmental friendly (renewable and recyclable/compostable) materials, is of great importance in order to eliminate an ecological crisis.

Examples of biofibres from Nordic agriculture and forestry are flax and hemp fibres and pine and spruce fibres. Industrial use of bio fibres such as wood fibres for the manufacture of pulp, paper, fibreboard and particleboard are well established in the Nordic countries. The renewed interest in industrial use of bio fibres within the automotive, building, furniture and packaging industries of European countries has led to development and production of many bio-fibre based products.

There is an abundance of biological fibres in the world, both from agriculture and forestry and there are many advantages of bio-fibre utilization. The use of bio fibres in industrial products is $CO_2$ neutral. The use of bio fibres is also very price competitive to synthetic fibres as fibre-glass. The bio-fibre based products show renewability and biodegradability.

The most beneficial advantage is the high specific strength and stiffnes. This advantage concerns mostly the long fibres from flax and hemp. These fibres show a low density and a relatively high tensile strength. These properties make together the bio fibres very competitive compared to glass fibres.

There are also disadvantages of bio-fibre utilization. One disadvantage is the variability of the fibre dimensions. The length and the diameter of the bio fibres vary very much in production. This is natural because we deal with biological material but the problem has to be solved in the production process.

Another disadvantege is the thermal degradation, when the biological material is exposed to fire with temperatures above 200 °C. However, from a waste point of view combustability is an advantage. Products can be disposed of through burning at the end of their useful service life and energy can simmultaneously be generated.

The main disadvantage of bio-fibre utilization is the lack of durability. The reason for this lack is the prone to water absorption. If no remedies are taken against the water absorption the effect is that bio-fibre based materials will not be dimensionally stable under changing moisture conditions. The tendency to absorb water can also be seen as an advantage because the fibres will biodegrade under certain circumstances through the actions of fungi and bacteria.

There are several methods to prevent water absorbtion of bio fibres. One method is acetylation. This is an environmental friendly method because no unnatural materials are supplied. With acetylation only the natural acetyl groups are strengthened. The acetylation gives both dimensional stabillity and protection against rot. Acetylized bio fibres may be used for the production of building materials as doors, windows and garden furnitures.

The lack of durability for bio-fibres in an alkaline environment may be possible to control by decreasing the alkalinity or by impregnation of the fibres. The use of bio fibres as reinforcement in cement based and concrete materials requires search for low alkalinity concrete.

When discussing durability and life cycle aspects on bio fibres the small use of chemicals in the cultivation and the production process has to be pointed out. There are nowadays production methods, which require small amounts of chemicals to strengthen the composite products but the utilization of bio fibres for the manufacture of industrial products in the future will be with as little environmental impact as possible.

## 5 CONCLUSIONS

There is an abundance of biological fibres in the world, both from agriculture and forestry and there are many advantages of bio-fibre utilization. The greatest advantage is the high specific strength and stiffnes. This advantage concerns mostly the long fibres from jute, flax and hemp.

The main disadvantage of bio-fibre utilization is the lack of durability. The reason for this lack is the prone to water absorption and the strong degradation of the fibres in an alkaline environment. Some of the disadvantages may also be considered as advantages. A good example of that is the biodegradability, which is caused by the water absorption. As the main disadvantages of bio-fibre utilization may be controlled at an extra cost by a number of known treatments it can be concluded that the advantages should exceed the disadvantages in utilization of bio fibres in composite products.

The goal for the bio-fibre composite product development should be to present for the market completely “green” products based only on biological raw materials. This is a challenge for research institutes and industry, which include solving the durability problems.

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# Failure Of Prestressing Steel Induced By Crevice Corrosion In Prestressed Concrete Structures

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**Summary: The application of prestressed concrete technology in civil structures is relatively recent. Therefore, the existing prestressed concrete members in such structure are still relatively young, and the corrosion and the concrete deterioration problems associated with this type of concrete members have became clearly evident only in the early 1980s. Damage during service, frequently appearing after several years or decades of exploitation, is usually most consequential. The lacking or insufficiently alkaline protection already from the beginning or its loss due to carbonation and/or depassivation after chloride attack are the major causes of later damage or even of a failure. Steel wire strands, due to their structure, usually consisting of steel wires helically laid about a core centre, frequently another wire, may be subjected to crevice corrosion even if they are well grouted with uncarbonated mortar but in presence of a little amount of chloride ions. Crevice can occur also in cable to duct (metal or plastic ducts) contacts. Crevice condition easily promotes a drastic change in local environmental composition, that is a pH reduction and an increase of chloride concentration. In such a condition steel strand failure due to hydrogen embrittlement can occur. Experimental results obtained in calcium hydroxide solution with added potassium chloride in simulated crevice condition on stressed steel strands, showed crack formation after only 1250 hours. Chloride concentration was lower than the critical amount to induce pitting corrosion or generalized corrosion at that pH (12.4), whilst was enough to induce extended crevice corrosion. In some case crevice attack was so intense to reduce drastically steel wire cross section.**

**Keywords: Prestressing steel, corrosion, crevice, cracks, failure**

## 1 INTRODUCTION

The application of prestressed concrete technology in civil structures is relatively recent. Therefore, the existing prestressed concrete members in such structure are still relatively young, and the corrosion and the concrete deterioration problems associated with this type of concrete members have became clearly evident only in the early 1980s. Although prestressed concrete members were generally manufactured with concrete of relatively higher strength, time has shown that they are subject to the same adverse effects of reinforcement corrosion as reinforced concrete members are. Documented cases of prestressed tension members failure as a result of corrosion make this a most pressing problem. Since prestressed concrete members rely on the tensile strength of the prestressing steels to resist loads, loss of even few wires or strands per member could result catastrophic.

Damage during service, frequently appearing after several years or decades of exploitation, is usually most consequential. The lacking or insufficiently alkaline protection already from the beginning or its loss due to carbonation and/or depassivation after chloride attack are the major causes of later damage or even of a failure. Responsible for that are usually the failures caused by shortcomings in planning and/or execution as well as inaccurate or inefficient structural measures. Execution faults and construction errors concern e.g. the injection of the ducts with mortar in case of post tensioned concrete (a mortar-free section of prestressing steel is exposed to the risk of corrosion, when penetration of moisture is possible and oxygen can enter the duct space ), the concrete technology (too small concrete cover and too low concrete quality, under certain conditions not protect prestressing tendons or prestressing steel), the procedure (technology) of the production of the structural elements, as well as waterproof sealing (not present or damaged) and drainage (damaged), that can lead to strong salt contamination of concrete, and very seldom the development of cracks outside and inside the coupling joint of prestressed bridges.

The up to then most spectacular damage in the prestressed concrete structures occurred in 1980. It was the collapse of the southern outer roof of the Berlin Congress Hall occurred 23 years after its constructing. The damage was primarily caused by corrosion-initiating and corrosion-promoting conditions in a part of the roof structure (Rehm *et al*. 1981).

Probably the most frequent occurrence of corrosion related problems on bridge structures as a consequence of planning and execution errors, is on post-tensioned precast segmental bridges. The first serious problem with corrosion of bonded post-tensioned bridges occurred in U.K. in the mid-1960 when the Bickton Meadows footbridge collapsed in Hampshire (Concrete Society 1996). This structure was a segmental post–tensioned construction type, and collapsed without warning under its own self-weight after only few years of service. Collapse occurred as a result of severe corrosion of the top tendons.

Then, in the 1980s, two bridges were found with serious problems: the Taf Fawr Bridge on the A470 in Wales and the Angel Road Bridge on the A406 North Circular in London. Soon, problems had been identified in other bridges, the most serious being the collapse of the single-span segmental post-tensioned Ynys-y-Gwas bridge in Wales in 1985 (Woodward and Williams 1988). The collapse of the bridge was due to corrosion of longitudinal tendons at the segment joints. The mortar at the joints was highly permeable and allowed moisture, chlorides, and oxygen ready access to the tendons. The structure was 32 years old with no evidence of distress prior to failure.

Finally, studies by the British Department of Transportation concluded in 1992 that there was currently no method to guarantee the grouting process and further work was needed. It was on 25 September 1992 that the Department of Transportation effectively banned post-tensioned grouted duct techniques from UK bridge construction. The moratorium was partially lifted only in the 1996 with the exception of segmental bridges constructed from precast segments, because there were still some reservations about the corrosion protection of the tendons where they cross the joints (Lewis 1996).

The UK was not the only place with problems. The post tensioned Melle Bridge built across Schelde in Belgium in 1956 collapsed early in 1992. The collapse was traced to corrosion of the ducted post-tensioning wires and, most worryingly, the bridge had been inspected, load tested, given a clean bill of health, re-waterproofed and restored to service just two years before (Concrete Society 1996).

Two very recent bridge collapses can be related to corrosion of prestressing steel: Saint Stefano bridge collapse in Italy in 1999 (Proverbio & Ricciardi 2000) and Lowe’s Motor Speedway footbridge in North Carolina (USA) in 2000 (Goins 2000).

Failure of prestressing steel in prestressed concrete structures is usually induced by corrosion. The most simple failure case is present when the high-strength steel fails because its notched-bar tensile strength in the region of corrosion pits has been exceeded. In such a case the failure has a brittle character. In practice such a pure brittle failure in case of prestressed concrete it is relatively seldom. The necessary condition of notched-bar tensile strength being smaller than the actual steel stress requires namely pit depth of at least 1 mm.

Fracturing of prestressing steel in structures is predominantly attributed to hydrogen- induced stress corrosion cracking (Nurnberger 1997). Hydrogen formation and absorbtion on steel surface is promoted by different corrosion conditions:

- activation or depassivation of certain regions of steel surface;
- localized corrosion (pitting corrosion);
- presence of certain substances (so-called promoters), which hinder the recombination of the atomic hydrogen to molecular hydrogen (such as sulphur, arsenic, thiocyanate and selenium compounds, which are in practice frequently found in concrete constructions).

Studies on behaviour of prestressing steel (cold drawn eutectoid steel) in $NaHCO_3$ solution, simulating carbonated concrete, was carried out by Alonso *et al*. (1993), showing that such steel is very susceptible to stress corrosion cracking (SCC) in carbonate-bicarbonated environment as a function of bicarbonate content.

Steel wire strands, due to their structure, usually consisting of steel wires helically laid about a core centre, frequently another wire, may be subjected to crevice corrosion even if they are well grouted with uncarbonated mortar but in presence of a little amount of chloride ions. Crevice can occur also in cable to duct (metal or plastic ducts) contacts. Crevice condition easily promote a drastic change in local environmental composition, that is pH reduction and increase of chloride concentration. In such a condition steel strand fracture due to hydrogen embrittlement can occur. Aim of this work is to investigate, by deep microscopy investigation, the evolution of corrosion phenomena on stressed cold drown steel wires subjected to crevice condition in calcium hydroxide saturated solution in presence of chloride.

## 2 EXPERIMENTAL METHODS

The material used was a 3/8” seven steel wire strand in cold drawn and stress relieved state. The chemical composition and properties are described in Table 1

**Table 1. Chemical composition and properties of the steel used**

| Chemical analysis (wt%) | | Mechanical properties | |
|---|---|---|---|
| Carbon | 0.81 | Ultimate tensile strength | 1940 MPa |
| Silicon | 0.22 | 1 % proof stress | 1750 MPa |
| Manganese | 0.67 | % elongation | 5.5 |
| Phosphorus | 0.007 | | |
| Sulphur | 0.004 | | |
| Copper | 0.09 | | |
| Chromium | 0.05 | | |
| Nickel | 0.05 | | |

Two different testing conditions were used. “Single wire” condition and a “bundle” condition. In the first case as received single steel wires (central wires extracted by a strand) with a diameter of 3.19 mm and a length of 150 mm were subjected to bending condition on glass reinforced plastic beams as shown in Figure 1. Wire deflection was imposed in order to reach in the middle section of the wire a stress equal to 0.8 time the UTS. Wires were kept bent by means of plastic strips, crevice condition were obtained by inserting a Teflon ring in the middle section of the wire. Samples were then immersed in a calcium hydroxide saturated solution (pH 12.4) closed to air (solution T), potassium chloride was added to the solution up to a concentration of 0.2 M. In such a condition it was obtained a $OH^-/Cl^-$ ratio less than the critical value to induce pitting corrosion (Andrade *et al*. 2001), but a chloride concentration sufficient to allow crevice corrosion occurrence. An electrical connection shielded by epoxy resin coating was made on each wire in order to monitoring electrochemical potential during the time.

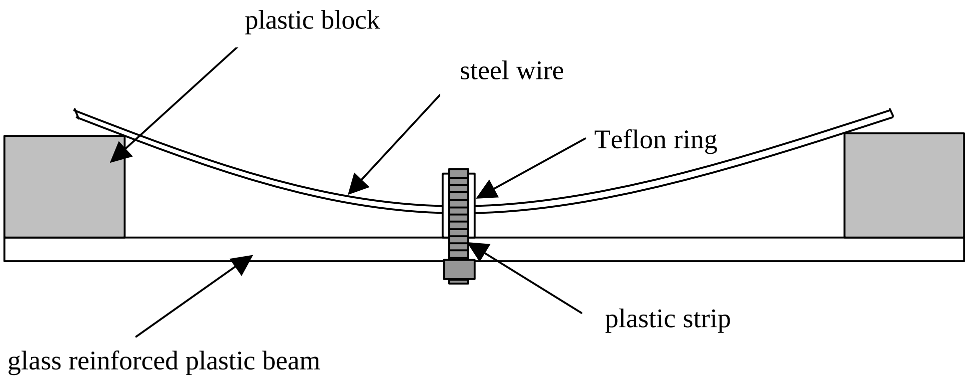


**Figure 1. Scheme of steel wire loading and crevice induction set up**

As a reference, in order to evaluate hydrogen embrittlement susceptibility, identical samples were immersed, at room temperature, in the solution A indicated in the ISO/DIS 15630-3 (Steel for reinforcement and prestressing of concrete – Test methods – Part 3: Prestressing steel). Test conditions are summarized in Table 2.

**Table 2. SCC susceptibility test condition after ISO/DIS 15630-3**

| Solution composition | Test condition |
|---|---|
| 5 g of $SO_4^=$ (as $K_2SO_4$), 0.5 g of $Cl^-$ (as KCl) | Constant axial load equal to 0.8 UTS |
| and 1 g of SCN- (as KSCN) for 1 litre of solution | 50±1 °C |

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Test in the bundle condition were performed by immersion in the solution T portion of seven steel wire strands, obtained by cutting 250 mm long pieces form the as received prestressing cable. Each bundle was secured on the extremities by plastic strips to avoid its opening. Such tests were performed to evaluate crevice occurrence between steel wires in condition similar to in field application.

## 3 EXPERIMENTAL RESULTS

Free potential versus time of steel wire during the tests is reported in Figure 2. After stabilization, and depending on crevice corrosion initiation time, free potential values ranged between –540 and – 470 mV (vs SCE). Such values lied well above the limit indicated by the equation proposed by Andrade *et al.* (2001) for critical chloride content for pitting initiation (Figure 3)

$$E_p = 310 \cdot \log\left[Cl^- / OH^-\right] - 15 .$$

No corrosion attacks were in effect evidenced on free steel surface after solution removal. Immersion test were ended after about 54 days (1280 h) even if no complete steel wire failures were observed. After removal of Teflon ring strong crevice attack was observed on some samples. Before samples observation corrosion products were removed by means of Clarke's solution (ASTM 1981).

A typical crevice attack was shown in Figure 4. Where crevice attack was particular intense complete disruption of steel occurred, corrosion being strongly influenced by steel drawing direction. No evidence of other type of corrosion damages were observed on such areas. Where crevice attack was less pronounced transversal cracks on steel surface were observed. Cracks were comparable for extension, and in some case for penetration depth, to those one observed on sample tested in solution A (thiocyanate solution) as shown in Figure 6.

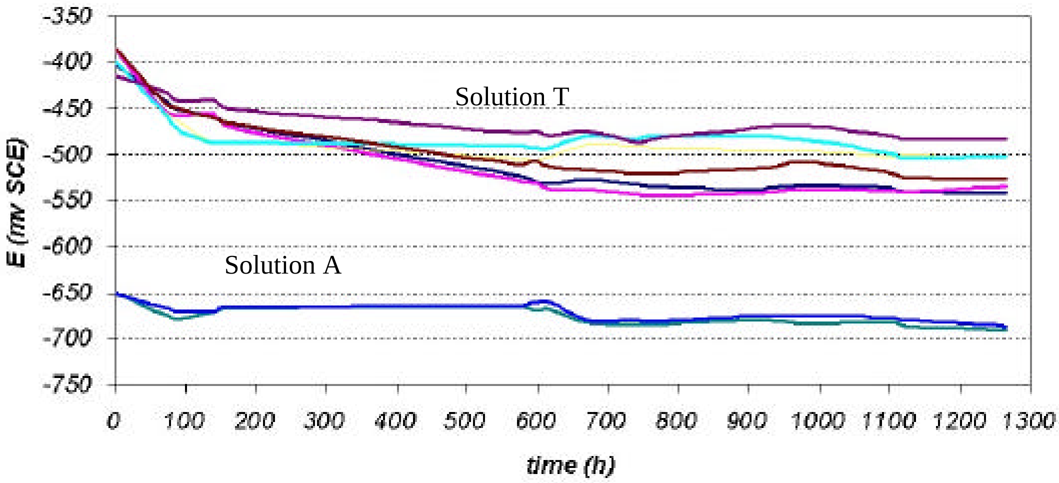


**Figure 2. Free corrosion potential of steel wires in solution T and A.**

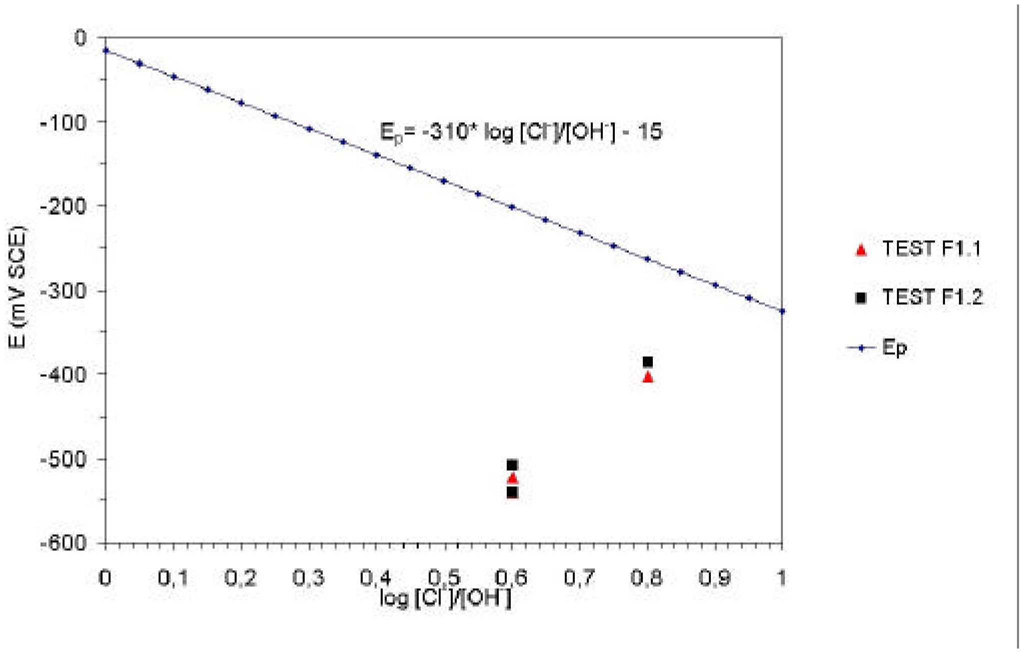


**Figure 3. Free corrosion potential of steel wires in solution T vs log[Cl-]/[OH-]**

No corrosion attack was observed on free steel surface of wire strand (“bundle” condition) as removed from solution T, unbinding steel wires revealed crevice corrosion development on contact surface between inner wires, corrosion attack being concentrated on an helical line following wires disposition (Figure 7). Strands were however not subjected to bending condition due to difficulties encountered in applying a sufficient stress to the samples without causing wire opening, so it was not possible to evaluate the influence of stress on evolution of corrosion attack for such type of samples.

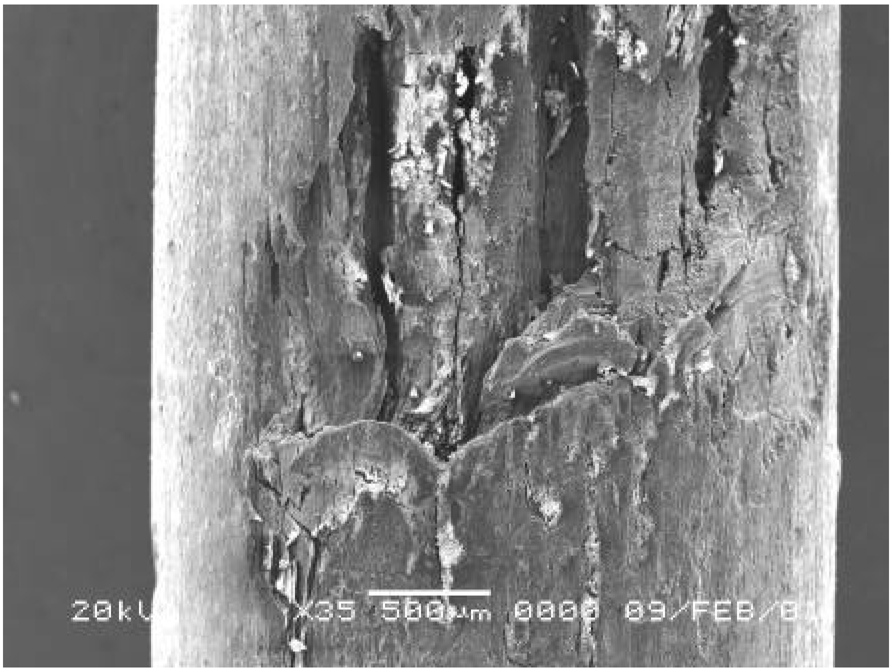


**Figure 4. Crevice corrosion attack under Teflon ring.**

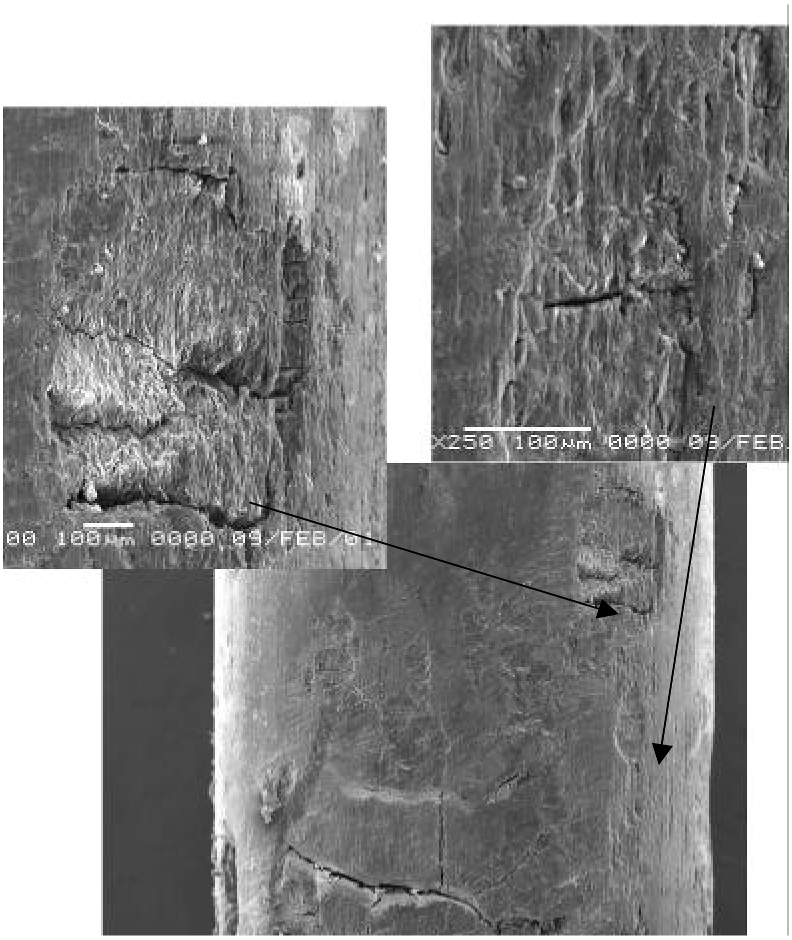


**Figure 5. Cracks localized in corroded areas**

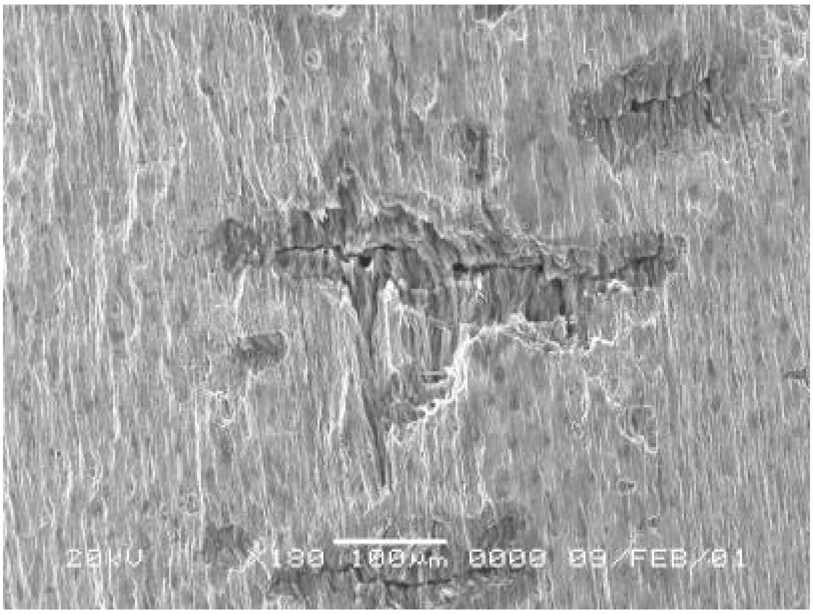


**Figure 6. Cracks on steel surface of sample immersed in the solution A**

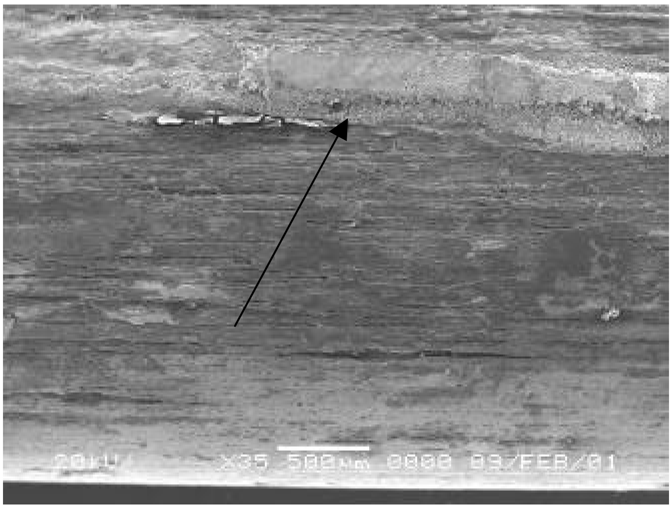


**Figure 7. Corrosion attack on the central wire of a seven wire strand after testing for 1100 h in the solution T**

## 4 DISCUSSION

Failure of prestressing steel is generally induced by corrosion. When corrosion attack is generalized (it occurs for examples in carbonated concrete or when steel is exposed directly to the atmosphere) failure occurs when cross section reduction, due to steel consumption, leads to an increase of stress in steel that overpass its UTS. It is well known however that prestressing steel is susceptible to stress corrosion cracking (SCC) (ACI Committee 222 2001; Alonso *et al*. 1993) and to hydrogen embrittlement (HE) (Mietz 1998). In some case brittle fracture is induced by localized corrosion attack, such as pitting (acidification at the pit tip allows hydrogen reduction and, as a consequence, hydrogen embrittlement of the steel). Cracks formation was in fact observed in some case at the bottom of pits (Nurnberger 1992).

Relatively few failures in the literature are attributed to brittle mechanism such as SCC and HE. One possible reason is that the prestressing steels normally used in prestressed concrete construction resist this type of failure quite well if we except special sensitive prestressing steel used during the '50s and the '60s in Germany (Nurnberger 2000). A number of problems have occurred on prestressed concrete structures in recent years that are not reported in literature. This probably because the failures have generated litigation with closure of trial proceedings or nondisclosure agreements between litigant. Another possible reason is that failures may have occurred in conjunction with pitting corrosion. In this case, the investigators may not realize that the failure are due to brittle HE because of the heavy pitting damage that may be present (ACI Committee 222 2001).

Brittle behaviour of steel wires extracted from heavily corroded prestressing cable taken from a prestressed concrete bridge was observed by Vehovar *et al.* (1998). It was supposed that low pH carbonated grouting mixture (pH ranging from 11.3 to 11.6) and the high amount of chloride present (1.1 wt % of concrete) allowed intense pitting corrosion to occur.

Cherry and Price conducted tests with two different strain rate on smooth, cold drown, stress-relieved prestressing wire (1800 MPa UTS) to determine if sodium-chloride solutions of varying pH and anodic polarization would cause SCC (Cherry and

Price 1980). Wire fractured on both tests. Cherry and Price assumed that since failure of the wires were by yield at the point where the cross-section has been reduced by pitting, hydrogen embrittlement does not lead to a synergistic interaction between stress and corrosion so that failure may be ascribed simply to pitting or crevice corrosion caused by the presences of chlorides.

Results reported in this work on the other hand, even if not conclusive, confirmed the evolution of hydrogen induced cracks (HICs) on steel surface in the acid environment of crevice, their growth was presumably strongly influenced by the stress condition which concentrate maximum stress on a limited section of the wire. It has to be stressed however that the major damage of steel wire was caused by the strong acid attack which interested more than the half section of the wire and that, for this type of prestressing steel, could be considered the main cause of the incipient wire failure. Further work on different type of prestressing steel (considering more hydrogen sensitive steel) was therefore planned for the future.

## 5 CONCLUSIONS

The results shown in the paper suggested that crevice condition, that can easily occur in prestressing steel in concrete construction due to execution faults or construction errors, may lead to severe corrosion attack. Crevice can also occur in chloride contaminated concrete on inner wire surface of wire strands.

Hydrogen induced cracks can develop in such condition on steel wire. On the basis of the results here reported it was however still difficult to evaluate the influence of such cracks on the failure of prestressing steel.

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# Investigation On The Corrosion Of Zinc Sheeting Of The Highly Insulated Cold Zinc Roofs In A Moderate Humid Region

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**Summary: During the last decades, with increasing concerns on energy efficiency and global warming, the U-factor of building elements has been pushed towards low threshold values. The trend to lower U-factor may greatly change the hygrothermal situation of traditional cold zinc roofs. Expectedly, that change could affect the corrosion of zinc sheeting in traditional cold zinc roofs.**

**In order to evaluate the hygrothermal performance and durability of well-insulated zinc roofs, a long-range field test program is carried out since November 1996. A total of four cold zinc roofs with high insulation quality (U<0.25 W/m$^2$/K) were constructed on the campus of KU Leuven, Belgium. Commercial zinc sheeting of 0.65 mm thickness (VM-zinc titanium) was used.**

**Investigations were conducted on the corrosion of zinc sheeting of the zinc roofs. The results showed: (1) The underside corrosion of the zinc sheeting was severe and far more serious than the upside corrosion. (2) Pitting was the main form of corrosion of the zinc sheeting. (3) A more perfect air-vapor tight retarder and a good ventilation of the roof cavity could not guarantee an elimination or a reduction of the underside corrosion of zinc sheeting in ventilated zinc roofs.**

**Keywords: Metallic roofs, corrosion, thermal insulation**

## 1 INTRODUCTION

Zinc roofing is common in Europe and may provide satisfactory service lives in urban atmospheres. It is usually considered to be maintenance-free, have a long life span, and easily adaptable to various design styles ranging from traditional to modern. The good performance of traditional zinc roofs is mainly attributed to the excellent resistance of zinc to corrosion and the favorable micro-environment surrounding the zinc sheeting.

Traditional cold zinc roofs were constructed with low insulation quality (most of them U>0.6 W/m2/K) and good ventilation. When carefully designed and installed, the condensation problem at the underside of zinc sheeting could be avoided or the condensate on underside surface of the zinc sheeting could dry out quickly. The underside corrosion of the zinc sheeting was not a serious problem. Traditional cold zinc roofs therefore performed well and served for a long time. Normally, a traditional cold zinc roof had a life span of over 30~40 years in industrial areas and of over 100 years in rural areas. In typical urban areas, a life span of 50~60 years could be expected.

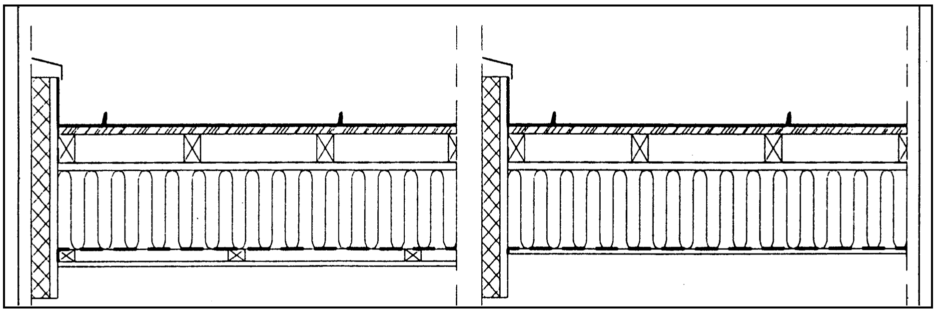
Nevertheless, in the last decades, the growing concerns about global warming and energy efficiency pushed the U-factor of building elements towards low threshold values. This trend to lower U-factors may greatly affect the performance of zinc roofs due to a worse hygrothermal environment surrounding the zinc sheeting. The lower U-factors in fact may result in stronger undercooling of the zinc sheeting and in lower air temperature in the roof cavity. This will decrease the removal of water vapor from the roof cavity by ventilation, which is usually considered to be a good solution to avoid condensation at the underside of zinc sheeting. In the worse cases, the phenomenon of undercooling may be so strong that the temperature of the zinc sheeting may drop below the dew-point temperature of the surrounding air. This makes the surrounding air to become a moisture source so that the effect of ventilation shifts from drying to wetting the zinc sheeting. Therefore, cold zinc roofs with high insulation quality may have a serious problem of corrosion at the underside of the zinc sheeting. Unfortunately, little research has been conducted on that problem.

In order to investigate the hygrothermal performance and durability of well-insulated zinc roofs, in November 1996, a long-range field test program was started in the VLIET test building of the Laboratory of Building Physics of the K.U.Leuven. This paper presents some results of this field study.

## 2 EXPERIMENTAL DESIGN AND EXPOSURE CONDITIONS

### 2.1 Experimental design

A total of four cold zinc roofs with high insulation quality ($U < 0.25$ W/(m²·K)) were constructed on the campus of KU Leuven, Belgium, in 1996. The dimensions of the roofs were about 1.8 by 3 m. The roof slope was 5 %. The orientation of two roofs was southwest (SW1c and SW2c), of the other two northeast (NE1c and NE2c). Of the four cold zinc roofs, NE2c and SW2c had a correctly installed air-vapor retarder, the others (NE1c and SW1c) a poorly installed air-vapor retarder. The section of the zinc roofs is shown in Fig. 1. The details are presented in table 1. In order to simulate the installation of electrical connections, a perforation of 47 mm in diameter through the gypsum board and the air-vapor retarder was made in the roofs SW1c and NE1c (hereafter referred SW1c and NE1c as air-open roofs, the other two zinc roofs without a perforation being called air-tight roofs) respectively. The perforation was located at the center of the air-open zinc roofs.



Air tight (SW2c, NE2c) Air open (SW1c, NE1c)

**Figure 1. The cold zinc roofs**

**Table 1 The cross sections of the cold zinc roofs (from outside to inside)**

| *Cold zinc roofs: air- tight (SW2c, NE2c)* | *Cold zinc roofs: air-open (SW1c, NE1c)* |
|---|---|
| • zinc sheeting with standing seams | • zinc sheeting with standing seams |
| • pine boarding 3/4" | • pine boarding 3/4" |
| • rafters 38×58 mm with ventilated cavity | • rafters 38×58 mm with ventilated cavity |
| • vapor open underlay foil | • vapor open underlay foil |
| • mineral wool 15 cm between rafters 60×175 mm | • mineral wool 15 cm between rafters 60×175 mm |
| • polyethylene (0.2 mm) air-vapor retarder | • polyethylene (0.2 mm) air-vapor retarder with a perforation ∅ 47 mm |
| • painted gypsum board | • painted gypsum board with a perforation ∅ 47 mm |

### 2.2 Exposure conditions

The weather data were automatically recorded every ten minutes in the weather station and transported to a computer. The main weather parameters recorded were wind speed, wind direction, solar radiation, air temperature, and relative humidity. The temperature, relative humidity, wind speed and wind direction were measured at a height of 9 m above ground level. The radiation was measured at the southwest edge of a paved roof. Both the inside and outside monthly average climatic information for the exposure experiod are presented in table 2.

**Table 2 Outside and inside climatic parameters during the exposure period of 28 months.**

| *Month* | *Outside* | | | | *Inside the house* | |
|---|---|---|---|---|---|---|
| | *Temperature* *°C* | *Relative humidity* *%* | *Solar gain (horizontal surface)* *$W/m^2$* | *Wind velocity* *m/s* | *Temperature* *°C* | *Relative humidity* *%* |
| Dec-96 | 0.7 | 81 | 20 | 1.7 | 22.6 | 42 |
| Jan-97 | -0.4 | 83 | 30 | 1.2 | 22.6 | 43 |
| Feb | 7.3 | 78 | 48 | 2.9 | 22.5 | 49 |
| Mar | 8.9 | 81 | 93 | 1.6 | 21.8 | 56 |
| Apr | 8.6 | 69 | 164 | 1.6 | 22.3 | 48 |
| May | 13.7 | 73 | 208 | 1.6 | 22.6 | 55 |
| Jun | 16.5 | 74 | 202 | 1.5 | 23.3 | 57 |
| Jul | 17.7 | 80 | 195 | 1.0 | 23.4 | 65 |
| Aug | 21.2 | 76 | 195 | 1.1 | 25.6 | 58 |
| Sep | 14.7 | 80 | 139 | 0.9 | 22.6 | 62 |
| Oct | 10.5 | 81 | 80 | 1.4 | 22.6 | 51 |
| Nov | 7.8 | 88 | 33 | 1.7 | 22.4 | 50 |
| Dec | 6 | 85 | 20 | 2.4 | 22.5 | 49 |
| Jan-98 | 5.3 | 81 | 27 | 2.6 | 22.5 | 43 |
| Feb | 6.5 | 80 | 61 | 1.5 | 22.9 | 42 |
| Mar | 8.3 | 79 | 80 | 2.1 | 22.9 | 45 |
| Apr | 9.9 | 81 | 125 | 1.9 | 22.9 | 47 |
| May | 15.8 | 73 | 197 | 1.3 | 22.6 | 43 |
| Jun | 16.8 | 79 | 197 | 1.4 | 23.4 | 52 |
| Jul | 16.5 | 82 | 142 | 1.1 | 23.0 | 60 |
| Aug | 14.7 | 86 | 126 | 1.2 | 23.2 | 49 |
| Sep | 15.7 | 86 | 107 | 1.3 | 22.6 | 63 |
| Oct | 10.5 | 88 | 47 | 2.0 | 22.5 | 53 |
| Nov | 4.4 | 88 | 38 | 1.5 | 22.8 | 40 |
| Dec | 5.5 | 88 | 19 | 2.1 | 22.2 | 44 |
| Jan-99 | 6.4 | 84 | 30 | 2.4 | 22.8 | 44 |
| Feb | 3.8 | 87 | 53 | 1.9 | 23.1 | 43 |
| Mar | 8.2 | 81 | 97 | 1.9 | 23.0 | 49 |

### 2.3 Collection of zinc specimens

One large zinc specimen was taken from each zinc roof in March 1999. The dimensions of the specimens were 50 by 70 cm. Each specimen contained a standing seam. All were visually examined on the spot. The location of the collected large zinc specimens on the zinc roofs is shown in Fig. 2.

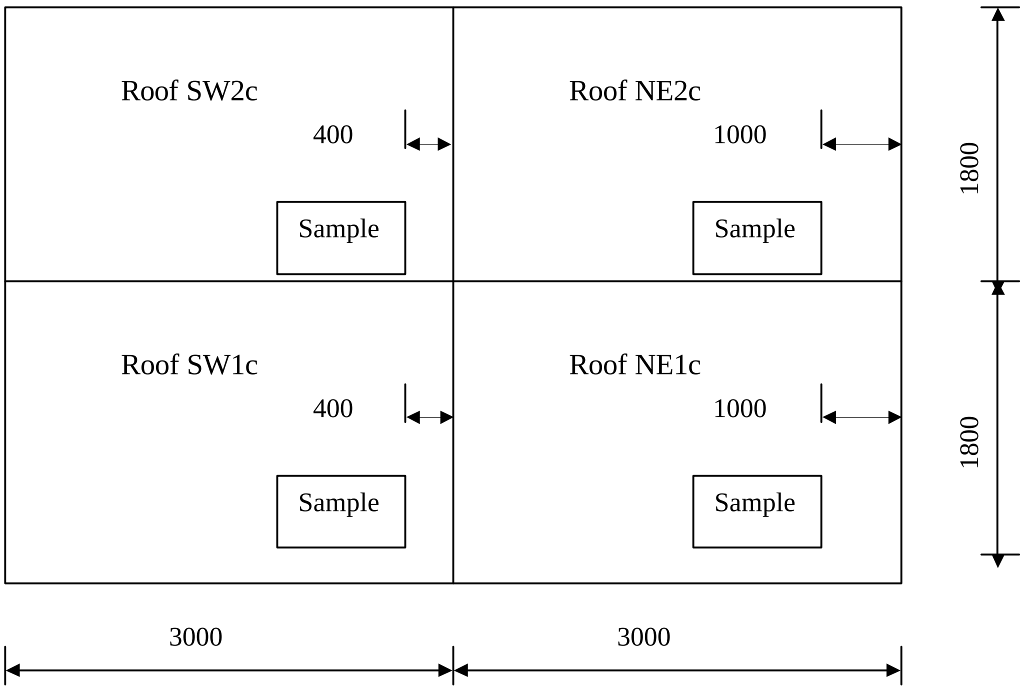



**Figure 2 The location of the collected large zinc specimens in the cold zinc roofs (top view)**

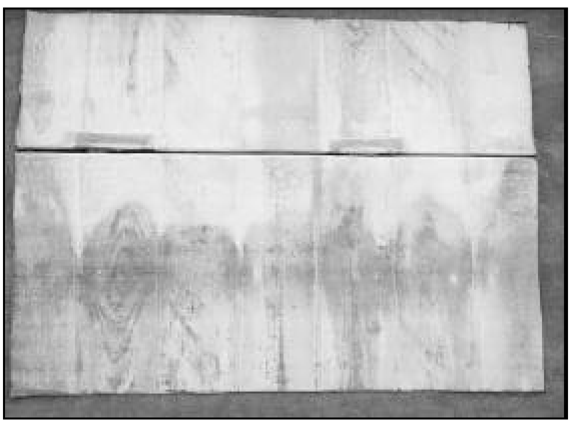
### 2.4 Cleaning method

In order to evaluate the corrosion type and the corrosion rate of the sheeting, some small samples were cut from the large specimens. These small samples were cleaned chemically as proposed by ASTM Standard G1-90: Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens (ASTM Designation: G 1-90, 1994). The samples were first immersed in warm (70 °C) ammonium chloride solution (10 %) for 5 minutes, rinsed in reagent water, and scrubbed with a light brush. Then, an ultrasonic cleaning machine (BRANSON-1200) was used to remove the remaining corrosion products from the deep pits. The above procedure was repeated two times. Removal was confirmed by examination with a low power microscope (5 to 50 ×). After the final treatment, the zinc samples were thoroughly rinsed and immediately dried.

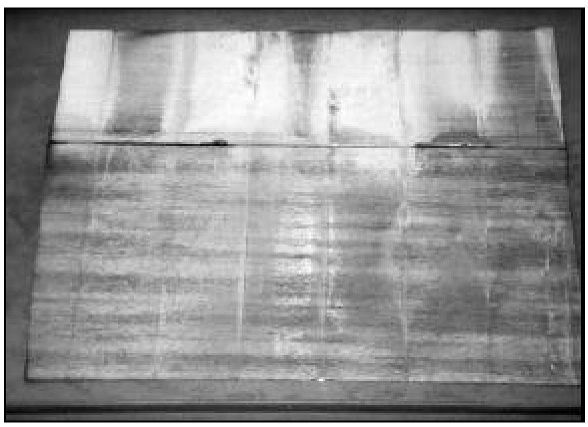
## 3 CORROSION TYPE OF ZINC SHEETING

### 3.1 General description of distribution of corrosion products

As shown in Fig. 3, the distribution of white corrosion products was not uniform at the underside of the zinc sheeting. It is also noted that there was no much apparent difference in corrosion between the air-open cold zinc roofs (SW1c and NE1c) and the air-tight cold zinc roofs (SW2c and NE2c). Generally, in light of the distribution of the corrosion products, there existed two typical regions on the underside of the large specimens. One region was heavily covered with white corrosion products (here labeled type I), the other was covered by a thin white film of corrosion products (here labeled type II). The directional characteristic of rolled zinc sheeting can clearly be seen on the areas with a thin white film of corrosion products.



Roof NE1c



Roof SW1c

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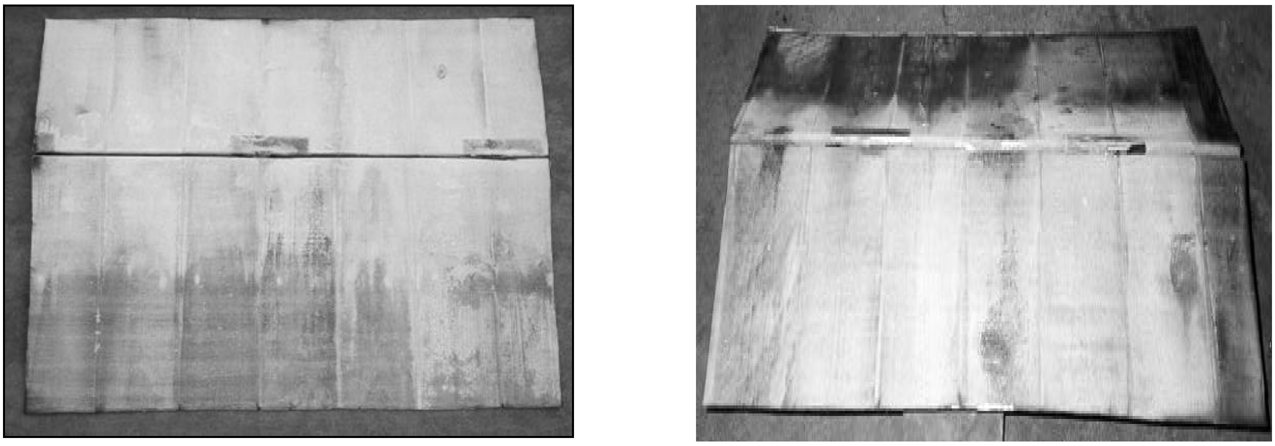

Roof NE2c Roof SW2c

**Figure 3 Large zinc specimens taken from the cold zinc roofs (50´70 cm)**

Figure 4 illustrates a wood scar on the wood deck in NE1c and the appearance of the corrosion product on the zinc surface corresponding to the scar. We believe that the underside surface of the zinc sheeting somewhat touched the wood scar due to the sign of the scar on the zinc surface. Examining it carefully, it is found that the center area of the wood scar was a little higher than the edge area. The height gradually reduced along the annual ring of the scar from the center to the edge. So, if the zinc sheeting contacted the wood scar, the most possible contacting place was the center area. Making a comparison between them, it is noted that the corrosion of the zinc sheeting corresponding to the center area of the wood scar was lighter than that corresponding to the edge area of the wood scar. This may partly demonstrate the influence of the contact situation between the zinc sheeting and the wood deck on the appearance of corrosion products on the zinc surfaces. For our cases, we may say that the corrosion pattern on the zinc sheeting contacting the wood deck or having much small gaps between the zinc sheeting and the wood deck took the type II, while that in the no-contact areas took the type I.

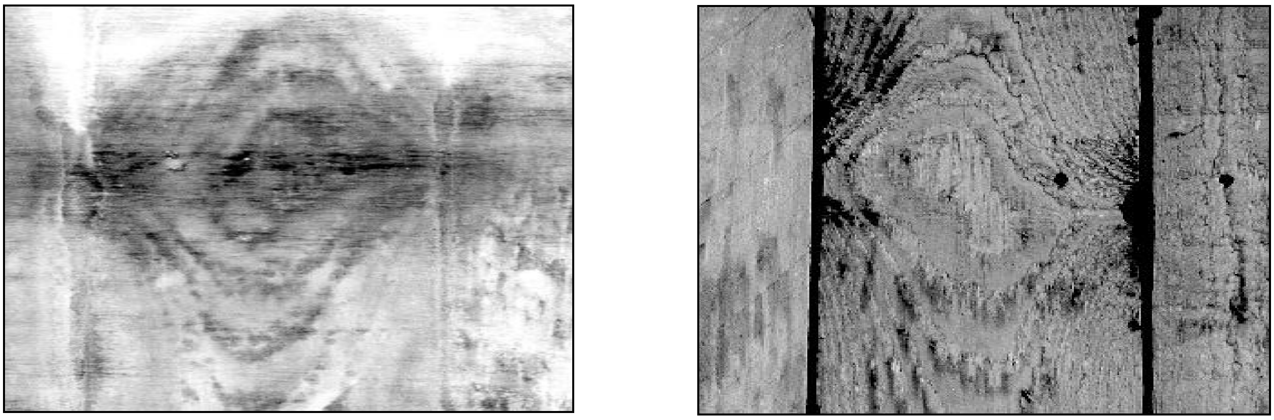

**Figure 5 Area of the zinc sheeting contacting a wood scar (left) and the corresponding wood scar in the wood deck (light)**

### 3.2 Type of corrosion

The types of corrosion that are commonly found to occur on zinc sheeting can be divided into four groups, namely general corrosion, pitting corrosion, galvanic corrosion, and intergranular corrosion. Of the corrosion types, general corrosion is most common.

Figure 6 shows the typical appearances of the underside of the zinc sheeting in the no-contacting areas after removal of the corrosion products. Clearly, a lot of big pits developed on the surface. The distribution of the pits was roughly uniform, but the dimension had some variation. Figure 7 illustrates the typical appearance of the underside of the zinc sheeting in the contacting areas after removal of the corrosion products. The zinc surface developed some dull appearance and the pits were very small. The distribution of these small pits was quite uniform. The corrosion at the no-contacting areas was much more serious than that at the contacting areas. The pits in both the no-contacting areas and contacting areas generally took a circular shape.

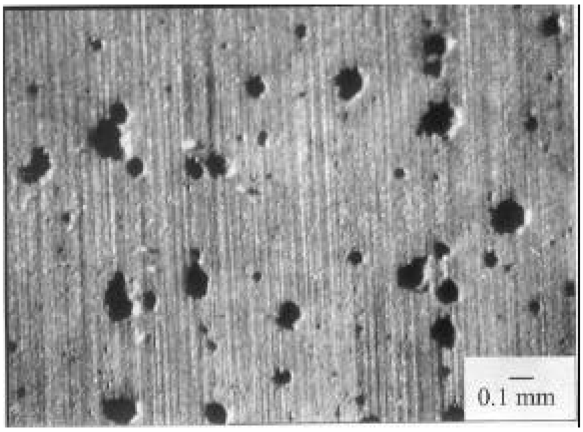


**Figure 6 The typical appearance of the underside of the zinc sheeting in the area not contacting the wood deck after removal of the corrosion products**

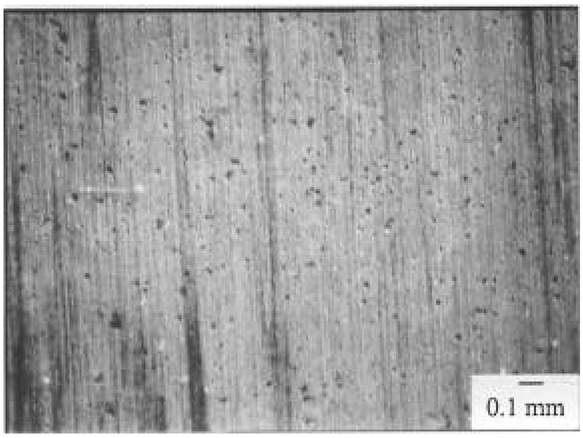


**Figure 7 The typical appearance of the underside of the zinc sheeting in the area contacting the wood deck after removal of the corrosion products**

For the upside of the zinc sheeting, the typical appearance after removal of the corrosion products is shown in Fig. 8. It is obvious that numerous small pits developed over there. The distribution of the pits is quite uniform from a macroscopic point of view. The pits seem shallow and many of their depths might be smaller than their diameters.

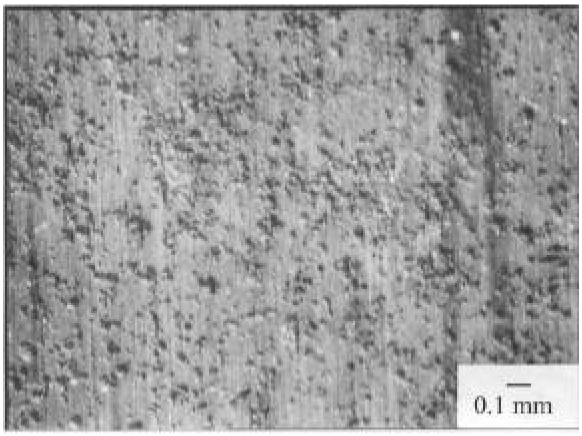


**Figure 8 The typical appearance of the upside of the zinc sheeting after removal of the corrosion products**

## 4 CORROSION RATE OF ZINC SHEETING

### 4.1 Method of evaluation

It is well know that pitting corrosion can be more serious than general dissolution attack. The growing rate of pit depth is dependent on both the environmental and the metallurgical conditions. It is very important to be able to determine the extent of pitting corrosion, either in a service application where it is necessary to predict the service life of zinc roofs, or in laboratory test programs that are used to select the best structures for zinc roofs.

In evaluating pitting corrosion, both the average corrosion rate based on mass loss and the average pit depths cannot readily be used. The corrosion estimation based on mass loss can seriously underestimate the corrosion penetration caused by pitting. For the corrosion estimation based on the average pit depth, there are also some problems. First, many smaller pits become too indistinguishable to be measured due to stopping growth very soon after initiated, and second, the deeper pits are of most practical importance.

For metallic roofs, the deepest pits will initiate leakage if they continue to propagate. Therefore, it is not the overall corrosion rate or the number of pits that are of interest, but the deepest pits since they cause the failure of the roof. Of the available evaluating methods, the extreme value statistical theory has been proven to be a powerful tool to analyze maximum pit depth data. This theory can be used to estimate the maximum pit depth of a large area of material on the basis of the examination of a small portion of it. Thus, we used the statistical theory of extreme value to evaluate the corrosion rate of the zinc sheeting.

### 4.2 Collection of maximum pit depths

As shown above, the corrosion situations in the contact areas and no-contact areas were quite different. Considering the quite light extent of pitting corrosion in the contact areas, we only evaluated the extent of pitting corrosion in the no-contact areas.

A total of ten small samples (5×5 cm) were collected from each area under concern on the large zinc specimens. The small samples were cleaned chemically using the method described previously. Five possible deepest pits on each sample were located by examination under a low-power microscope (5 to 50 ×) and were circled with a pencil. Then, the circled pits were measured using a powerful microscope "Quick Vision" (200 to 1000 ×). A pit was centered under the objective lens at low magnification, which was increased until the pit area, including the lips, filled most of the field of vision. The microscope was first focused on the uncorroded surface at the lips of the pit and then on the lowest part at the bottom of the pit. The difference between the initial and the final reading was counted as the pit depth. On a single pit, the measurement was repeated three times and the largest value was selected as depth of the pit. Of the depths of the possible five deepest pits on each sample, the largest one was considered to be the maximum pit depth of that sample. The measured maximum pit depths for each zinc roof based on the ten small samples are given in table 3.

**Table 3 The measured maximum pit depth of each zinc roof based on the ten small samples**

| *Roof* | *SW1c* | *NE1c* | *SW2c* | *NE2c* |
|---|---|---|---|---|
| Maximum depth (μm) | 167 | 180 | 273 | 228 |

### 4.3 Type of distribution of the maximum pit depths

Table 4 presents the results of a statistics analysis on the measured maximum pit depths. Since the maximum depths at the upside of the zinc sheeting were too small to be located and measured, they are not considered in the analysis. As shown in Table 4, all Fs are far less than 0.01. This shows that the linear regression relationships between the reduced variates and the maximum pit depths are highly significant. This can be taken as a proof that the maximum pit depths are sampled from parent populations of the exponential type and that the infinite population of maximum pit depths follow the Gumbel distribution.

**Table 4 Equations of linear regression between the reduced variates and the maximum pit depths in the cold zinc roofs**

| Roofs | Equations of linear regression | R square | F (significance of F-test) |
|---|---|---|---|
| SW1c_nca | y = 0.031943*x - 3.09361 | 0.969 | 2.63E - 07 |
| NE1c_nca | y = 0.059071*x - 8.33586 | 0.954 | 1.21E - 06 |
| SW2c_nca | y = 0.023750*x - 4.19073 | 0.962 | 5.62E - 07 |
| NE2c_nca | y = 0.042821*x - 7.96814 | 0.921 | 1.09E - 05 |

Note: "nca" — no-contact areas.

### 4.4 Estimation of the maximum pit depth on the zinc roofs

It is interesting to know how many maximum depth pits deeper than a specific value may exist in a zinc roof. Since the number of the maximum depth pits is area dependent, both the contact area and the no-contact area should be known for this analysis. Nevertheless, there is some difficulty of obtaining the exact areas of the contact surface and no-contact surface for the entire zinc roofs in practice. On the basis of the appearance of the large zinc specimens, we assumed that the ratio of the contact area to the no-contact area was 0.4:0.6, without regard to the very little pits in the contact areas. Thus, the cumulative probability of the maximum pit depths can be expressed as

$$1-\frac{m}{N_c}=0.6\times\exp\left(-e^{-\frac{x_c-u_{nca}}{\alpha_{nca}}}\right)+0.4 \qquad (1)$$

Where:

m — the number of maximum depth pits deeper than a specific value

$N_c$ — the total number of small samples for each cold zinc roof.

$\alpha_{nca}$ and $u_{nca}$ — The scale value and mode value of the distribution of the maximum pit depths in the no-contact areas

$x_c$ — the specific maximum depth, μm.

Rearranging the equation (1), the specific value $x_c$ that there are m maximum depth pits deeper than $x_c$ can be expressed as

$$x_c = u_{nca} - \alpha_{nca} \ln\left(\ln\left(\frac{0.6N_c}{0.6N_{c-m}}\right)\right) \quad (2)$$

Table 5 summarizes the values of $x_c$ obtained from equation (2) for m = 1, 10, 50, and 100. It is evident that, for all cases in question, the values of the pit depths in SW2c and NE2c (the two roofs with a correctly installed air-vapor retarder) were much larger than those in SW1c and NE1c (the two roofs with a poorly installed air-vapor retarder). This means that the maximum pit depths in SW2c and NE2c were much deeper than those in SW1c and NE1c. For the comparatively deep maximum depth pits (i.e., m = 1 and 10), the maximum pit depths in SW1c and SW2c were deeper than those in NE1c and NE2c, respectively.

**Table 5 The values of $x_c$ in the cold zinc roofs (unit: mm) when m = 1, 10, 50, and 100.**

| Number of pits | SW1c | NE1c | SW2c | NE2c |
|---|---|---|---|---|
| 1 pit | 321 | 262 | 479 | 353 |
| 10 pits | 249 | 223 | 382 | 299 |
| 50 pits | 198 | 196 | 313 | 262 |
| 100 pits | 176 | 184 | 283 | 245 |

### 4.5 Estimation of the failure probability of the zinc roofs

Perforation of zinc sheeting is one of the most important factors in determining the service life of zinc roofs. The first perforation, whose time and location is a matter of probability, is a useful indicator to reveal the situation of the zinc sheeting. It is very important to know the probability of failure. In the following analysis, we consider the zinc sheeting to fail once there exists a maximum depth pit deeper than the thickness of the sheeting.

The failure probability of the no-contact areas of the individual zinc roof $P_{nca}$ may be written as

$$P_{nca} = 1 - \exp\left(-e^{-\frac{x_t - u_{nca} - \alpha_{nca}\ln 1296}{\alpha_{nca}}}\right) \quad (3)$$

where:

$x_t$ — The thickness of zinc sheeting in microns, 650 µm.

Considering the very small pits in the contact areas, we may assume its failure probability $P_{ca}$ to be zero. The events $P_{nca}$ and $P_{ca}$ are independent. Thus, the failure probability of a cold zinc roof is given by

$$P_c = 1 - \exp\left(-e^{-\frac{x_t - u_{nca} - \alpha_{nca}\ln 1296}{\alpha_{nca}}}\right) \quad (4)$$

Table 6 summarizes the failure probability of the cold zinc roofs. As seen in table 6, the failure probability in SW2c and NE2c is obviously higher than that in SW1c and NE1c, respectively. It is also noted that the probability of failure in SW1c and SW2c is clearly higher than that in NE1c and NE2c, respectively.

**Table 6 Failure probability of the zinc roofs**

| *roof* | *SW1c* | *NE1c* | *SW2c* | *NE2c* |
|---|---|---|---|---|
| Failure probability | 2.77E-5 | 1.14E-10 | 1.70E-2 | 3.04E-6 |

## 5 DISCUSSION

It was expected that no severe corrosion was found on the upside of the zinc sheeting in the zinc roofs because a specific protection layer developed there as time progresses. This layer contains zinc hydroxide and basic zinc salts and is called patina. It is this layer that prevents the upside of the zinc sheeting from being seriously attacked. Although numerous very little pits developed on the upside surface, they normally are not a big problem. Pitting corrosion in atmospheric environments has seldom been reported as the main cause of failure of zinc sheeting. Pitting corrosion is normally induced in clean rural environments. While the depth of the pits increases with time, the ratio of pit depth to surface-average corrosion penetration decreases (Zhang, 1996). In an ASTM atmospheric exposure program, pits were observed on high-grade zinc and on a 1% Cu-zinc alloy after two years of exposure in four different environments (Showak and Dunbar, 1982; Viart, 1984). In general, the upside pitting corrosion of the zinc sheeting may have little impact on the service life of the zinc roofs in question.

Normally, contact between zinc and hygroscopic materials, especially in moist conditions, may result in more severe corrosion. Obviously, as shown, it was found that the underside corrosion of the zinc sheeting was more severe in the no-contact areas than in the contact areas. This may be attributed to the strong undercooling of the sheeting.

The measured undercooling of the sheeting could be so severe that its temperature fell below the dew-point of the outdoor air for quite a long time. Due to the seams (about 5 mm) among laths in the wood deck, the ventilated air in the roof cavity might bypass the laths and reach the underside of the zinc sheeting. So, condensation induced by ventilation could occur in those non-contact areas during severe undercooling periods. That caused the underside corrosion of the sheeting.

For the areas contacting the wood deck or the areas with much small gaps between the sheeting and the wood, however, less condensation might be expected because of the difficulty of ventilated air reaching those spots and because of the hygroscopic inertia of the wood. The wood deck absorbs the little condensation if any. In these areas, the corrosion rate of the zinc sheeting mainly depends upon the moisture content in the wood deck. Its threshold value to induce zinc corrosion is about 20 % by weight (Building Research Establishment, 1986). Therefore, if the moisture content in a wood is continuously kept below that critical level, zinc does not suffer severe corrosion even contacting the wood. As the moisture contents in the wood deck during the last three years of measurement generally remained lower than 20 %, it is reasonable that less corrosion at the underside of the zinc sheeting in the wood-contact areas occurred in comparison with the no-contact areas.

It has been widely accepted that high inside air exfiltration and poor ventilation of roof cavity are the main factors resulting in severe underside corrosion of zinc sheeting in cold roofs. The precautions against the underside corrosion of the sheeting mainly focus on these two aspects. Dunbar and Showak (1982) proposed two basic measures to eliminate or reduce this type of corrosion: 1. Installing a good vapor barrier underlay material under the zinc sheeting to minimize the amount of water vapor that can permeate and condense against its underside. 2. Providing some airflow between the zinc sheeting and the underlay to rapidly dry any moisture that does condense. In order to increase the ventilation in roof cavity, Zaher (1995) recommended that zinc roof slopes should be more than 1 in 12, otherwise, additional venting is needed to assist the drying of the underside of zinc sheeting.

The ventilation in the roof cavity of the cold zinc roofs in the test building has been investigated by Janssens (1997). It was found that the relation between the ventilation rate (G, $m^3/m/h$) and the wind speed (v, m/s) was "$G = 27.8\ v$". This result showed that wind could induce a good ventilation rate in the cavity of the cold zinc roofs on the test building. Obviously, that good ventilation could not eliminate or lessen underside corrosion of the sheeting in these highly insulated cold zinc roofs. It is found unexpectedly that the underside corrosion of the zinc sheeting in SW2c and NE2c (air-tight zinc roofs) was more severe than that in SW1c and NE1c (air-open zinc roofs). Apparently, the low air exfiltration could not eliminate or even reduce underside corrosion of the sheeting in these highly insulated cold zinc roofs either.

A possible reason for the unexpected results is the strong undercooling of the sheeting. Zheng et al (1999) made an investigation on the hygrothermal behavior of the zinc roofs. The results showed that the severe undercooling of the sheeting could make the surrounding air to become a moisture source so that the effect of ventilation was changed from drying to wetting the underside of the sheeting. Under that situation, more air exfiltration combined with good ventilation of the cavity might increase the thickness of water layer at the underside of the sheeting. It is well known that the thickness of water film on zinc surface is one of the most important factors affecting the corrosion rate of zinc in atmosphere. Thicker water film decreases the diffusion rate of oxygen. In this situation, the thicker the water layer, the lower the corrosion rate was.

As shown, the traditional precautions (low air exfiltration and good ventilation in cavity) seem not to work effectively in cold zinc roofs with high insulation quality. New techniques and roofing systems should be developed, if we like to guarantee a significant reduction of underside corrosion in zinc roofs with high insulation quality.

## 6 CONCLUSIONS

A field investigation was performed to evaluate the corrosion type and corrosion rate of the zinc sheeting in the highly insulated cold zinc roofs in a moderate humid region. The following main conclusions were drawn:

1. Numerous very little pits developed on the upside of the zinc sheeting. The distribution of the pits was quite uniform. The upside of the zinc sheeting corroded very little.
2. At the underside of the zinc sheeting, pitting corrosion was the prevailing corrosion type.
3. There existed two different forms of underside corrosion of the zinc sheeting. In the areas where contacted the wood deck, its surface developed some extent of dull appearance and only somewhat pitting corrosion occurred; while in the areas not contacting the wood deck, numerous large and deep pits developed.
4. The low air exfiltration and the good ventilation in the roof cavity could not guarantee an elimination or even reduction of the underside corrosion of the sheeting.
5. Generally speaking, the corrosion of the sheeting was very severe in the highly insulated cold zinc roofs after being exposed for only three years.

6. New techniques and roofing systems should be developed for highly insulated zinc roofs in moderate humid regions.

## 7 ACKNOWLEDGMENTS

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# The Influence Of Rain And Humidity On The Lifetime Of Transparent Polymeric Roof Materials

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**Summary: PMMA and PC undergo photochemical ageing leading to a gradual loss of material transparency and appearance during service life which is desired to reach 50 years or more. Available artificial weathering test methods are unable to distinguish materials in terms of the speed of ageing in outdoor use as roof materials. In order to design an appropriate accelerated or time compressed test method the "Worst Case" scenario of the respective climatic conditions must be found. Therefore, samples of PMMA and PC were exposed to artificial weathering tests using different climatic conditions in terms of relative humidity and surface wetness. In case of PMMA it was found that water accelerates the photo degradation enormously. The fastest ageing occurred when liquid water and UV radiation acted simultaneously on the sample surface and the PMMA reached its highest water content during weathering. In the case of PC it was found that both vapour and liquid water speed up the photoageing, similarly to PMMA. But differently from PMMA the "Worst Case" scenario for PC proved to be weathering conditions with high relative humidity. A test method has to be able to run artificial weathering under the different moisture conditions which can occur in the field.**

**Keywords: Polymeric roofings, poly(methyl methacrylate), Polycarbonate, photodegradation, weatherability**

## 1 INTRODUCTION

Transparent polymeric materials are increasingly used in the place of glass for transparent applications in the building sector because they are lightweight, tough, easily formed into complex shapes and transmit light well. However plastics are more susceptible to degradation by weathering than the traditionally used glass; notably to the combined effects of ultraviolet light, heat and moisture. A large range of plastics glazing is available ranging from highly photo stable materials that are commonly used without photostabilisers to poorly photo stable polymers that require extensive protection against the destructive weather conditions. Applications such as roof materials usually require a lifetime of several decades. Therefore, accelerated or time-compressed artificial weathering tests are used to assess the weatherability of plastics glazing or combinations of plastics glazing. Nowadays, the favoured method to design an appropriate test is the so-called "Test-tailoring". This means that each kind of plastic glazing needs its specially designed weathering tests. Test-tailoring requires the knowledge of the "Worst case" scenario of the weather periods occurring in the field, characterising the combination of environmental factors that most accelerate the photodegradation for the concerned plastics glazing. Today's standardised artificial weathering tests for plastics are performed mostly too dry and do not meet the "Worst case" scenario of most types of plastics glazing consuming a lot of time without showing the typical degradation phenomena. Therefore, our work is directed at the influence of water (vapour or liquid) during weathering on two widely used materials for plastics glazing (PMMA and PC). A good example of the use of PMMA glazing over about thirty years is the roof of the renowned Munich Olympic stadium. The roof plates had to be replaced completely in the last few years because they reached the end of their lifetime. The commonly observed failure mode in PMMA is the gradual development of crazes. Because the PMMA used was stretched in two axis the crazes developed in a horizontal direction leading to a scale-like splitting-off of material from the surface. The result of this degradation phenomenon was an unacceptable loss of light transparency. But none of the standardised artificial weathering tests used commonly for evaluating the material quality could cause this phenomenon in the laboratory. Therefore, the great variation in lifetime of the plates that was observed during the last three decades in the environment could not have been predicted. A special phenomenon that could be observed on the roof plates in the field led to the suspicion that moisture conditions had accelerated the deterioration of the PMMA. Figure 1 shows an almost 25 year old highly damaged roof plate. The almost clear surface parts which can be seen near the metallic disks or frames needed for fixing the plates were mostly warmer than the other parts in clear nights and therefore, affected by dew for shorter periods and less intensely. Lehmann (1997) describes the frequent changes of wetting and drying as the most likely cause for the damage of the weathered roofing site. But the contribution of water to the loss of fastness by photodegradation cannot be discarded. Finding experimental evidence for this presumption was one aim of this work.

The contribution of humidity to the photodegradation of polycarbonate is controversial. In artificial weathering studies, Pryde (1985) reported that pre-hydrolysis of polycarbonate accelerated photo-induced discoloration when compared to dry samples. However, the presence of high humidity levels during exposure to UV was reported to have an opposite effect, decreasing the rate of photo-induced yellowing according to Factor and Chu (1980) and Pryde (1984). While Tjandraatmadja et al (1999) reported that under moderate humidity (42%) the rate of yellowing underwent an acceleration stage in the early stages of degradation, but as exposure progressed dry samples experienced more intense yellowing. On the other hand, high humidity seems to favour the rate of photo-oxidation in polycarbonate according to researchers, such as Clark and Munro (1984) and Rivaton et al (1986). In context with developments to improve the relatively bad scratch resistance of PC by covering it with appropriate hard coats, more knowledge about the behaviour of the basic PC sheet material at extreme weather conditions is necessary. Further clarification of the influence of water on the photodegradation of PC due to the differences in the existing opinions was the other aim of this work.

## 2 EXPERIMENTAL

### 2.1 Materials

Two types of PMMA and four types of PC were evaluated. All of them were commercially available sheet materials. The identification of the samples is shown in Table 1.

**Table 1: Identification and features of the glazing examined**

| **Sample** | **Type** | **Thickness** | **Property** |
|---|---|---|---|
| PMMA 1 | GS 215 stretched | 4 mm | Uncoloured |
| PMMA 2 | GS 816 stretched | 4 mm | Light brown coloured |
| PC-1 | Lexan | 1 mm | Only basic stabilisation |
| PC-2 | Indoor | 3 mm | Only basic stabilisation |
| PC-3 | APEC HT | 3 mm | Only basic stabilisation, optical quality, |
| PC-4 | Longlife | 3 mm | Both sides co-extruded with highly stabilised PC films |

### 2.2 Methods

Artificial weathering was performed using a commercially available fluorescent UV lamp weathering device, model "Global UV Test" in combination with a special box which allowed to expose specimens to three different moisture conditions at equal temperature at the same time and in the same weathering chamber. The conditions are characterised in Table 2.

**Table 2. The three different weathering conditions**

| Short term | dry | humid | wet |
|---|---|---|---|
| Climatic condition | 50°C; 13% RH | 50°C; 98 % RH | 50°C; steady under a 5 mm sheet of water |

The test ran under constant conditions. The specimens were subjected to continuous UV radiation (39 W/m$^2$). Spectral radiation distribution was performed according to ISO 4892-3: 1994 (no UV output below the normal solar cut-off of 290 nm). The "Global UV Test" equipment enables the application of the test under defined and reproducible weathering conditions, which is a requirement for good repeatability and reproducibility. The temperatures of the panels were constant to ±1K. Periodically specimens were taken out of the weathering device and their properties (listed in Table 3) were determined.

**Table 3. Examinations of the weathered specimens**

| Glazing | Property | Measuring technique |
|---|---|---|
| PMMA | appearance | Visual (unarmed eye) |
| PC | Exposure up to first occurring of crazes | Visual (unarmed eye) |
| PC | Spectral transmission | Carl Zeiss – DMC 25 |
| PC | Haze | Gardiner Pivotal Sphere UX-10 |
| PC | Gloss 60° | BYK Gardner Tri Gloss meter |
| PC | Colour | BYK Gardner Color guide, sphere 65/10° (white tile background) |
| PC | Surface image | LAIKA macroscope M420/ DC200 |
| PC | Surface profile | WYKO white light interferometer |
| PC | SEM | Phillips XL30 at 2 kV |
| PC | Chemical change | Bruker FTIR |
| PC | Acid resistance | ADF test of the BAM |

## 3 THE EFFECT OF WEATHERING

### 3.1 PMMA

After certain exposure periods specimens of each material were taken out of the weathering device, dried at normal room conditions and photographed all using a scanner. The weathering was finished after 130 days and the last specimens were dried in vacuum to determine the water content. Weathered specimens of PMMA 1 are shown in Figure 2 . As can be seen in Figure 2 the test duration that is necessary for the first occurring of scales clearly depends on the climatic conditions. No visible deterioration could be found after weathering under dry conditions. The joint action of UV radiation and liquid water proved to be the "worst case" for PMMA. In this case the steady wetness of the surface led to the highest water content in the polymer. An environment with high relative humidity showed a slower deterioration obviously because of the lower reachable water content in the polymer. No difference could be found in the behaviour of the two PMMA qualities.

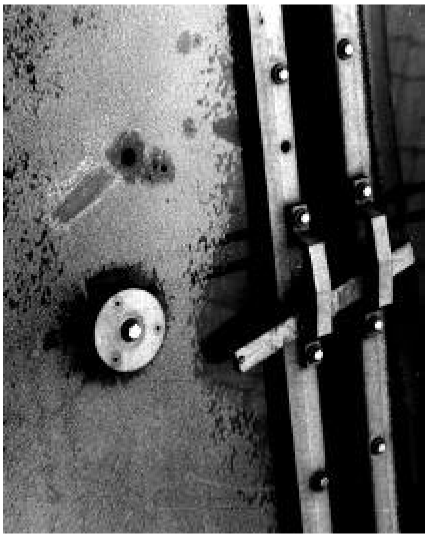
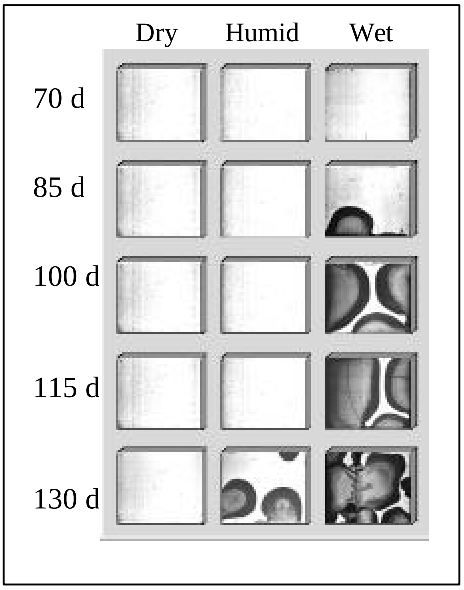


**Figure 1. Image of damage of a 25 years old roof plate made from PMMA 1**




**(<0,01 %) (1,5 %) (3 %)**

**Figure 2. Image of damage of PMMA 1 after exposure in dependence on the test conditions (from left to right) and the test duration from top to bottom. In brackets the water content after 130 days of exposure**

## 3.2 Polycarbonate

### 3.2.1 Spectral transmission

One of the characteristic traits of polycarbonate photodegradation is yellowing.

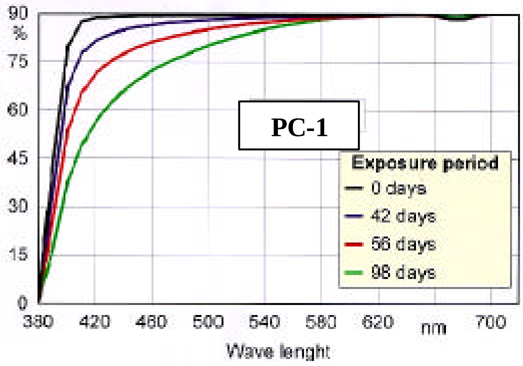



**Figure 3. Spectral Transmission of PC1 at wet weathering conditions in dependence of the exposure period**

The behaviour of PC-1 shown in Figure 3 is a typical example of the change of the spectral transmission of PC due to the weathering. The most evident variation could be found at the 400 nm shoulder resulting in yellowing. The transmission curves for the other test conditions and for the other samples are very similar. They differ only in speed and intensity of decrease of the 400 nm value. In the same context, the yellow/ blue chromaticity factor b* shifting in the yellow range predominates significantly in the results of the colour change measurement in accordance to CIE La*b* coordinates. Therefore, in the discussion concerning colour the influence of the weathering conditions on yellowing is presented using the factor b* only.

### 3.2.2 Colour change

As can be seen in Figure 4, all of the four PC samples are yellowing progressed at different rates during different stages of exposure.

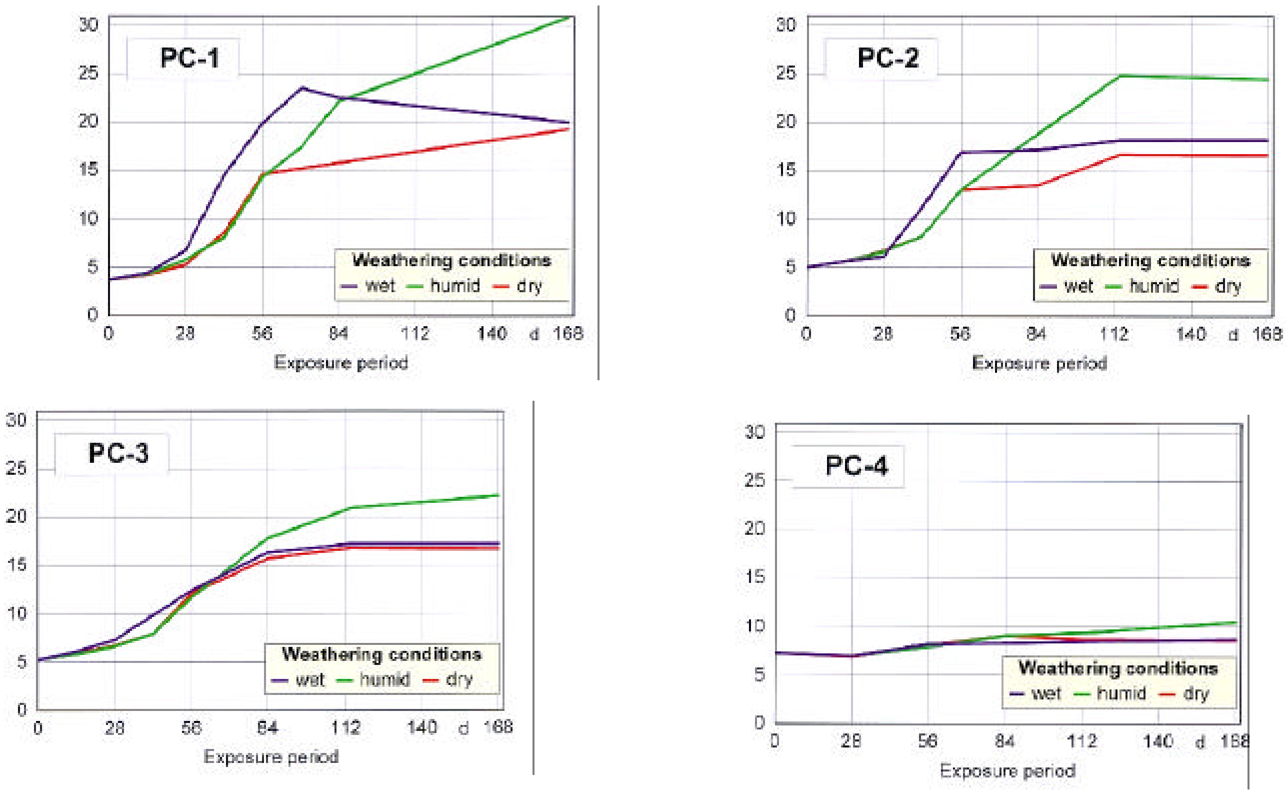



**Figure 4. Yellowing of polycarbonate upon exposure**

The increase in yellowing was not significant during the first 30 days of exposure. This could be due to the combined action of UV stabilisers present in the sheets' formulation and the induction period required for the concentration of yellow polyconjugated species to cause noticeable color changes. After the initial period the rate of yellowing increased at an accelerated pace between 50 and 80 days of exposure, after which it slowed down again at an apparent plateau. The rate of increase was dependent on the characteristics of each sample. PC-4 (Long life) experienced the least variation with exposure, hence it could be assumed to undergo the least photodegradation. It displayed less than 20% change for “Wet” and “Dry” conditions and a maximum change of 43.8% for the “Humid” one. In comparison the other types of polycarbonate displayed increases in yellowness ranging from 220 - 325% (PC-3), 230-400% (PC-2) and 420-735% (PC-1). Hence the better stabilisation system in the PC-4 samples protected them effectively from the effect of UV. PC-1 and PC-2 (both indoor quality) samples were the most susceptible to photodegradation as would be expected, since both would contain less stabilisers being indoor type materials. Moisture had a detrimental effect to the yellowing process. Samples exposed to UV and covered permanently with a water film (wet) yellowed at a faster pace than the other samples during the acceleration stage. However the yellowness values (b*) reached similar valuesas the samples degraded at 13%RH(dry) after prolonged exposure (>100 days). Whilst, the samples photodegraded at 98% RH displayed the most intense yellowness (b*).

### 3.2.3 Development of crazes

Crazes are another typical feature of photodegradation of polycarbonate. Figure 5 presents the WYKO-profiles of several surface sections of weathered PC-1 specimens showing the variety of weathering-caused alterations of the surface morphology:

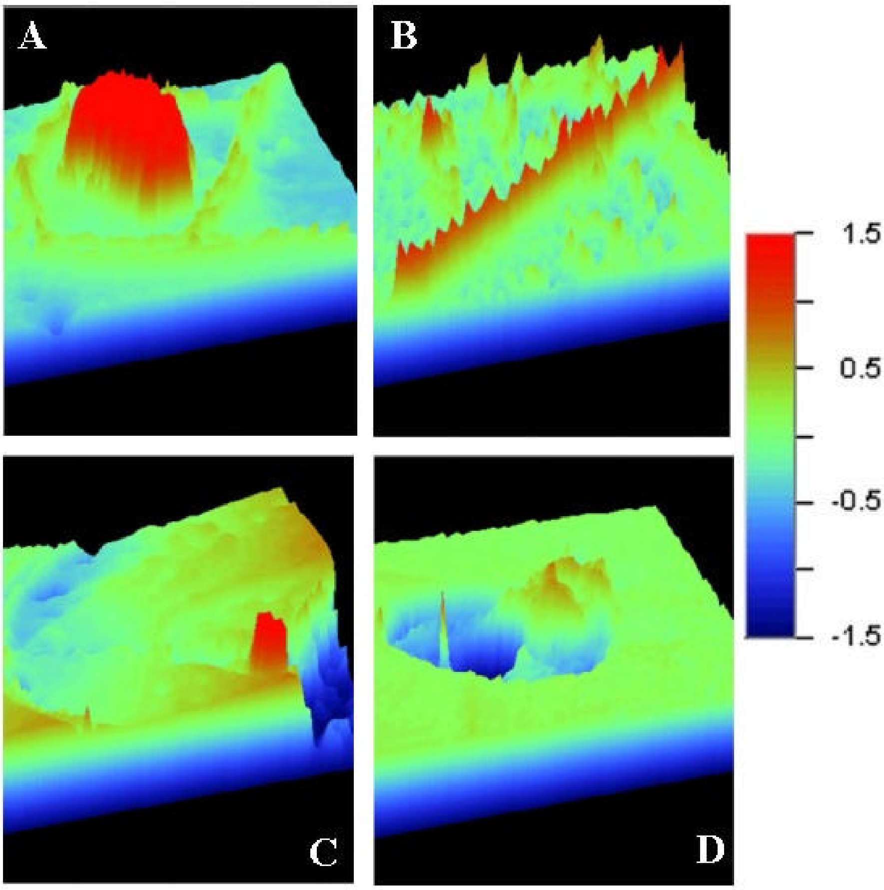



**Figure 5. Typical details of the surface profile of differently weathered specimens of PC-1 (A: 56 days, wet - 60 x 45 µm; B: 84 days, wet – 220 x 300 µm, C: 84 days, humid – 60 x 45 µm; D: 84 days, humid – 190 x 250 µm; height in µm)**

Blisters up to 1.5 µm in height surrounded by crazes up to 0.5 µm in height and up to 15 µm in width (see profile A). Crazes up to more than 1.5 µm in height (see profile B). Caves and holes down to deeper than 1.5 µm signalling loss of material (see profiles C and D). These features were distributed irregularly across the sample surface. But in general crazes dominated in comparison to other features. Therefore, only the development of crazes was used to discuss the weathering-related changes of

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surface morphology in the further discussion.  In contrast to the literature, crazes were risings predominantly and fissures beside the risings appeared only secondarily as can be seen in the SEM photos in Figure 6.  It was found that the development of crazes ran in close agreement to the change of colour characterised by the following arguments which were derived from the images of 2 mm wide surface areas of the differently weathered specimens in Figures 7 and 8: First: Visibility and density of crazes increased with increasing exposure period.  Second: The dry-weathered specimens did not show any crazes in general. Comparing the specimens weathered at moist conditions crazing is stronger on the humid-weathered specimens than on the wet-weathered ones after 168 days of weathering particularly in case of PC-1.  Third: In comparison of the different PC samples, PC-1 showed the strongest crazing and PC-4 had not shown any crazes after 168 days.  Fourth: Crazes first occurred during weathering under wet conditions (see also Table 4).

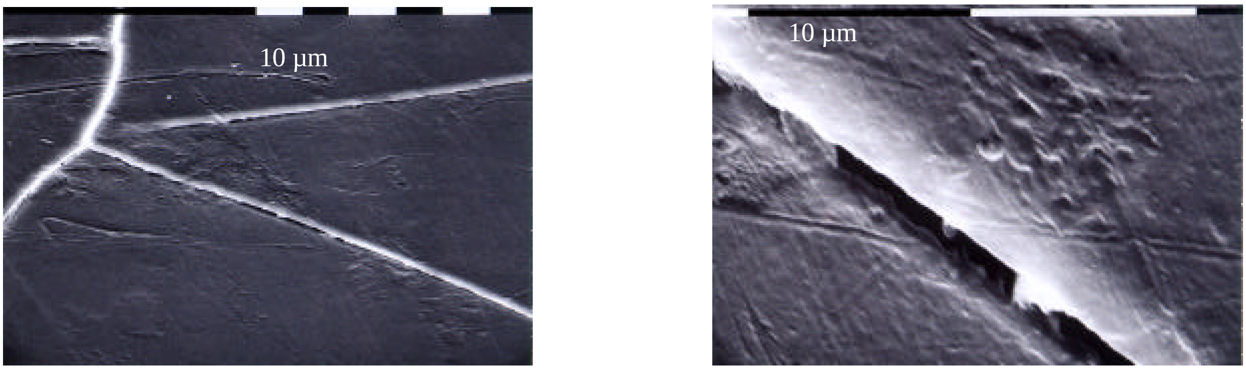


**Figure 6. SEM image of a section of the 56 days wet-weathered specimen of PC-1 in different magnification**

**Table 4.  Exposure period up to the first occurrence of crazes identifiable by the unarmed eye**

| **Sample:** | **PC-1** | | | **PC-2** | | | **PC-3** | | |
|---|---|---|---|---|---|---|---|---|---|
| Weathering condition | Dry | Humid | Wet | Dry | Humid | Wet | Dry | Humid | Wet |
| Exposure period for crazing (in days) | >168 | 56 | 42 | >168 | 56 | 42 | >168 | 70 | 42 |

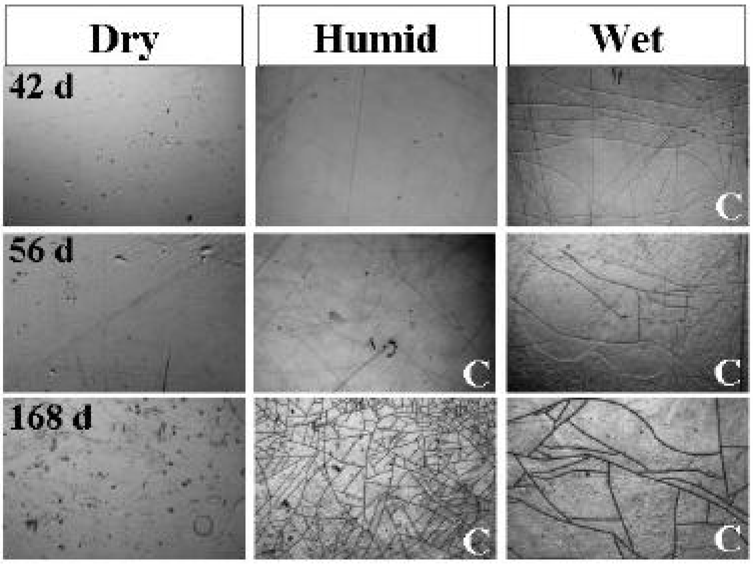


**Figure 7.  Crazing of PC-1 in dependence on weathering conditions and exposure period (width of the section: 2 mm, C means: Crazes can be spotted by unarmed eye)**

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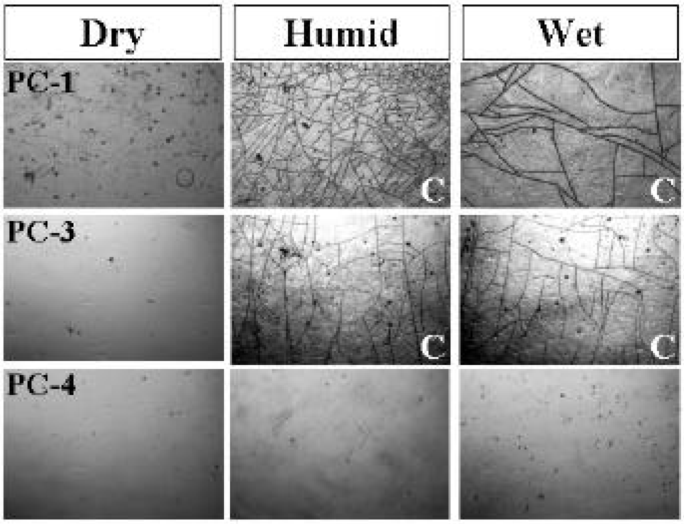


**Figure 8. Crazing after 168 days in dependence on weathering conditions and type of PC**

3.2.4 Changes in Gloss and Haze

Surface deterioration was reflected by gloss readings. As the roughness of the surface increased, more light scattering took place, reducing the gloss readings for the specimens. Haze, which measures the light scattered by the polymer, both internally and on the surface, increased almost linearly with exposure for all samples. All samples behaved similarly. Sample PC-1 best showed the influence of moisture (see Figure 9). It exemplifies the typical behaviour of all four samples, i.e. that changes in gloss and haze seemed to be independent of the moisture levels during the initial 50 days of exposure. Similarly to changes in colour or intensity of crazing, the samples that were exposed to dry conditions experienced a significantly slower change in gloss and haze than the humid or wet exposed ones with further weathering. In contrast to the phenomenon observed in yellowing and crazing, wet conditions resulted in stronger deterioration than humid conditions in terms of gloss and haze. Sample PC-4 which did not show any significant changes in yellowing and surface morphology due to moisture conditions during 168 days of exposure revealed its first ageing effects by small changes in gloss and haze at humid and dry conditions (see Figure 10).

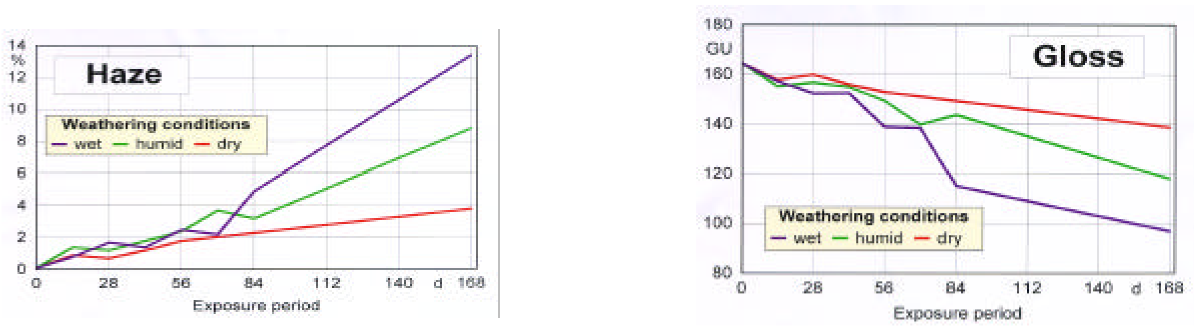


**Figure 9. Haze and gloss of PC-1 upon weathering depending on the weathering conditions**

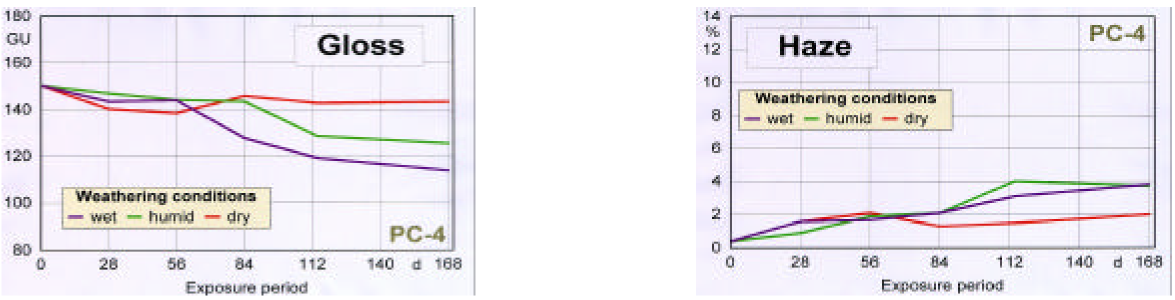


**Figure 10. Gloss and haze of PC-4 upon weathering depending on the weathering conditions**

3.2.5 Change in acid resistance

Acidic atmospheric precipitation like acid rain, dew and fog are products of the modern industrial society and an essential climatic factor that can accelerate the photodegradation of polymers. Roof materials used in highly industrialised or heavy traffic areas must also have sufficient resistance against the action of acidic precipitation. To assess the resistance of plastics glazing to acidic precipitation it is usual to test whether attacks by acidic deposition result in changes in appearance.

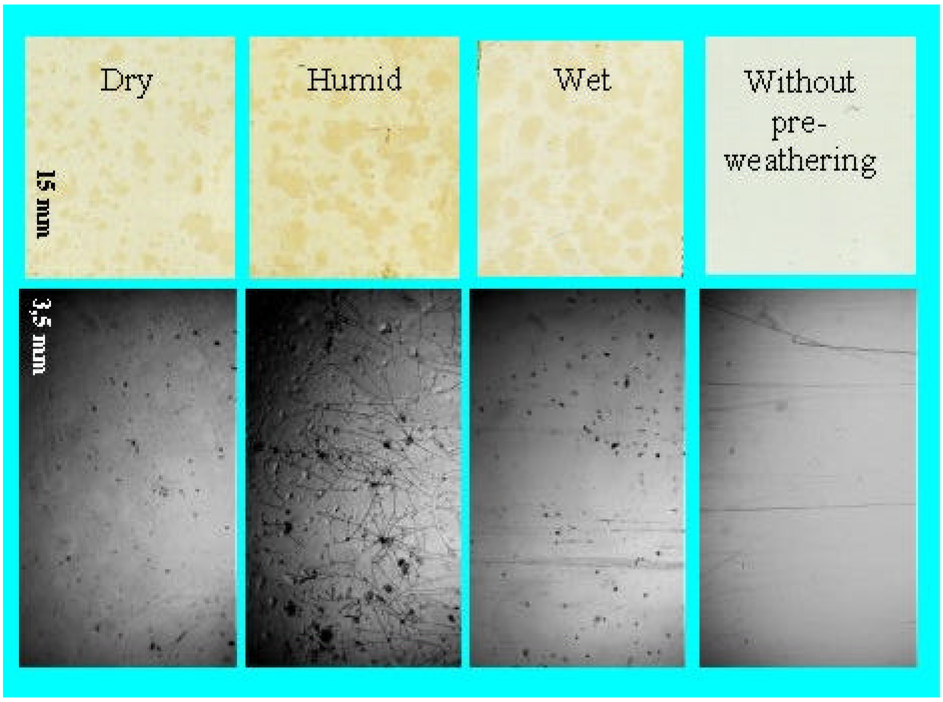


**Figure 11. Influence of the weathering conditions on the acid etch resistance of PC-4 (colour and surface image after 168 days of pre-weathering plus 5 ADF cycles in comparison to an un-weathered specimens after 5 ADF cycles)**

The Acid Dew and Fog test (ADF test in short) as described by Schulz (1999) was used to determine this material property. Because the specimens of sample PC-4 did not show any crazes or essential yellowing after 168 days of weathering under dry, humid as well as wet conditions yet, they were selected and subjected to the ADF test together with an un-weathered specimen. Figure 11 shows colour and surface images of these specimens after this test. In contrast to the unweathered specimen, the acid deposition (a solution of sulphuric, nitric and hydrochloric acid diluted to pH 1.5) which was sprayed on the specimen surface once a day caused varied strong yellowing on all of the three pre-weathered specimens and crazing on the humid weathered one, revealing that weathering had decreased the good initial acid etch resistance of polycarbonate differently with a strong and clear dependence on the weathering conditions. High relative humidity combined with high temperature proved clearly to represent the “worst case” scenario in this regard. According to this conclusion crazes could also be found on specimens of PC-4 that were exposed outdoors in Jacksonville, Florida. The Jacksonville harbour area is known to combine humid warm climate with frequent occurrence of acid rain and dew. On the other hand practical experiences with PC longlife proved that natural exposure at the temperate European climate characterised by lower absolute humidity and temperature did not generate any visible damage. Examples for the good durability are the more than ten years old roofs of the “BayArena” sporting stadium in Leverkusen and the main station in Krefeld, Germany. Long service life of PC-4 can also be expected under dry hot conditions.

### 3.3 Chemical changes

The deterioration experienced by the polycarbonate samples is the result of chemical changes due to photodegradation and physical processes. The photo-degradation reactions, Photo-Fries and photo-oxidation, are triggered whenever a sample is exposed to UV in air, and degradation is evidenced unless enough UV stabilisers are used in its formulation (Factor 1996, Rivaton 1995). These reactions result in chain scission and crosslinking, generate yellow species responsible for colour changes, form oxidised species (eg. alcohols, acids, esters, etc) (Factor et al 1987), alter molecular weight, etc. In summary they change the chemical and physical composition of the degraded layer. The extent of such photoreactions is limited by the depth of penetration of UV radiation into the polymer, which varies according to the source of radiation, material, exposure time, etc (Factor and Chu 1980).

Figure 12 compares the IR-spectra of PC-1 samples aged for 168 days under the different moisture conditions (Wet, Dry and Humid) with the control sample (without weathering). As highlighted in the hatched square the peaks in the carbonyl region

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(1600-1800 $cm^{-1}$) differed for each condition. At dry conditions the peak at 1775 $cm^{-1}$ decreased in intensity and a shoulder of new peaks from 1770 to 1600 $cm^{-1}$ was developed. This indicates the presence of -C=O from new photo-oxidised species, for example ketones, carboxylic acids, aromatic esters, which can absorb in that range. The other samples show a much smaller shoulder in that region, although the wet-weathered specimen had a larger shoulder than the humid-weathered one. This would suggest that the surface layer experienced more oxidation in the order Dry>Wet>Humid. Such observations confirm that moisture interferes with the photo-oxidation of polycarbonate.

### 3.4 The contribution of moisture to photodegradation of polycarbonate

Moisture aggravated the physical changes initiated by photodegradation. For example, the onset of crazing contributed markedly to the alteration of optical properties: It coincided with the rapid increase in haze, reduction of light transmission and gloss. Furthermore, moisture also interfered with the rate of the photodegradation reactions. The acceleration observed during yellowing suggests that moisture aided in the formation of yellow species. The exact mechanism is not yet known. There are two possibilities, moisture might favour the mobility of free radicals within the degraded layer and the transfer of products, or it could react with free radicals slowing down the photo-oxidation reaction. The results prove that the first possibility seems to be predominant. Additionally, moisture levels could also affect the rate of photo oxidation by limiting the diffusion of oxygen into the polymer. Another possibility is that stabilisers could have leached out of the polymer due to moisture. If loss of stabilisers took place then it would contribute to a lower resistance to oxidation in a wet environment compared to a humid one. In summery, the rate of degradation is a complex process that could be influenced by a number of factors, including loss of stabilisers, oxygen availability and mobility of free radicals in presence of moisture.

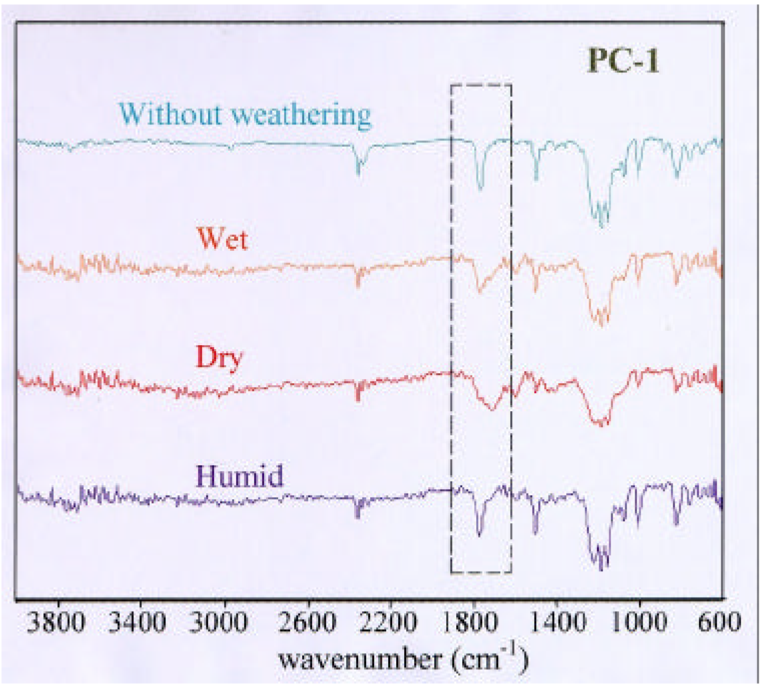


**Figure 12. Infrared spectra of PC-1 exposed to different moisture conditions (168 days)**

For example, as moisture condensates on the polymer surface, the area for diffusion of oxygen from air to the sample would decrease. This could explain why the samples which were covered by a layer of water during weathering (Wet) showed a slower degradation than those weathered at high humid conditions exposed constantly to a vapour phase containing about 20 % of oxygen.

## 4 CONCLUSIONS

The Photodegradation of plastics glazing currently used for transparent roofing made from PMMA, PC or combinations of both materials is accelerated by the presence of water during weathering. In the case of stretched PMMA plate material, the presence of liquid water during weathering or other conditions guarantying high water content in the polymer proved to be the “Worst Case” scenario of the weather conditions in practice. Chemical changes by hydrolysis promoted presumably by UV radiation seem to be more likely responsible for the decrease of mechanical fastness than the fatigue by the frequent changes in volume by wetting and drying as claimed in former publications (Lehmann 1997).

In the case of Polycarbonate humid-warm climatic conditions guarantying high contents of water and oxygen simultaneously in the surrounding vapour phase proved to be the “Worst Case” scenario. However the photo oxidation promoted by water in gaseous condition seemed to be the predominant reaction in comparison to hydrolysis, as to the exact mechanism by which water facilitates photo oxidation this will have to be investigated further.

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## 5 ACKNOWLEDGMENTS

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# Corrosion Of Metals - Mapping Of The Environment In Iceland

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**Summary**: **Iceland is an island in the middle of the Atlantic Ocean, at a latitude of 63 - 66 °N, and is in the main path of low pressure systems in this part of the world. The country is dominated by a mountainous central part (highest peak 2119 m) and glaciers, with lowlands mainly along the south and west coasts. The climate is therefore characterized by a wet and windy south-western part and a colder-dryer northern part. The corrosion environment ranges from a wet, chloride-rich area in south, to a dry and colder area in the north. In fact, the biggest desert in Europe, is in Iceland. In the windy environment, pollution is seldom a problem for people or materials. The corrosive environment is now being mapped by weathering of small test pieces at 15 different locations in Iceland; the test pieces are made of steel, zinc, aluminium, galvanized steel, and surfaces with different types of paint. The first-year results have been analysed, and show a spread in weathering for zinc between 1.4 - 3.9 mm and for steel between 1.5 - 36.1 mm . This paper discusses project design and presents first-year results of the study.**

**Keywords. Corrosion, metals, atmospheric corrosivity.**

## 1 INTRODUCTION

### 1.1 The country and climate

Iceland is an island in the North Atlantic Ocean. The middle is uninhabitable highlands, and the built environment, thus, mainly along the coast. The climate is marine-influenced and geographic location is in the route of frequent low pressure systems that move over the ocean. Accompanying the low pressure systems are prevailing wind directions of southwest to southeast and this, in combination with mountains and a few glaciers, results in wide climate variation within the country.

The climate is windy and moist, and precipitation is very common on the south coast. The highlands, especially north of the glaciers, is very dry, with the area north of the largest glacier, being the biggest desert in Europe. The energy use for heating and electricity is environmentally friendly, as it involves geothermal energy and electricity from hydro power stations.

Heavy industry is on a small scale (three factories so far are located near Reykjavik). The number of cars per capita in Iceland is very high and pollution from them is noticeable in the capital city, Reykjavik, on the few calm days each year. In periods of gentle southeasterly winds, polluted air from Europe comes to Iceland. However, the air is mostly clean, due to the windy climate and rather minimal local pollution. Information on air pollution in Iceland is limited and measurements are mainly done to assess the pollution from traffic in the capital.

### 1.2 The market

Almost all building materials, except cement and rock materials, have to be imported to Iceland, and this obviously applies especially to all metals. The annual import of metals for the building industry is approximately 310 kg/capita, and other annual imports (transport vehicles, etc) about 185 kg/capita. Of this annual import, approximately 370 kg/capita of metals end up in a corrosive environment of some kind. Metals are recycled to some extent or about 0.2 kg/capita.

In the building industry, metals are mainly used for reinforcing concrete, various types of claddings, for pipes, etc. Almost all roofs are covered with corrugated sheets of either steel or aluminium, and walls of older wooden houses are likewise commonly covered with corrugated steel. In the last years, cladding of concrete walls in new buildings has become more popular. Import of metal cladding materials each year amounts to about 700,000 $m^2$. Hydro power stations are placed at different locations around the country and it has proven effective to develop a wide-ranging network of high voltage lines to even-out electrical disturbance in the distribution net. The high voltage electrical cables and masts are in environments, which have very different corrosive characteristics. Some are quite near the sea, in constant salt spray, and others are in the highlands.

Iceland’s market is small and far away from other markets, so transport costs easily run high. In this context, and generally speaking, it is important to obtain the best longevity of materials as possible, e.g. by limiting corrosion of metals. Therefore, corrosion speed and corrosive agents must be evaluated. Mapping to show the background atmospheric corrosion rate is likewise essential to be able to follow changes over time, but also to be able to inform the market about how the conditions relate to other known parts of the world.

Corrosiveness of the environment can be estimated using two different methods: direct measurements, or evaluation of atmospheric agents in accordance with ISO 9223. This paper presents results from measurements of first-year corrosion and compares them to mapping of the climate.

## 2 CORROSION AND WEATHERING TEST PROGRAM

Samples sized 100 mm x 150 x 2 mm were mounted in fasteners of polymer materials on a backing panel of plywood, such that the distance from the backside of the sample to the plywood sheet is 17 mm with 15 mm between different sample materials. The samples are placed at 15 test locations, each rack with a mounting board of size 530x860 mm at a height of 3 - 4m above ground, oriented to face south at an angle of 45° in accordance with EN ISO 8565. The test pieces are of technically pure, low carbon steel (C=0.05%) , zinc-coated carbon steel, zinc (> 99,9% pure), aluminium (1050 A and AlMg3) and different types of painted steel samples. Plans are to demount samples for evaluation after 1, 3, 5 and 10 years. The measurements consist of recording the corrosion of the metals and evaluating the condition of paints. The project started in 1999, and therefore, results to date are limited to one year’s corrosion.

The test locations are shown on Figure 1, and numbering of them is listed in Table 1.

The results for corrosion rate of steel and zinc after one year are shown in Table 1.

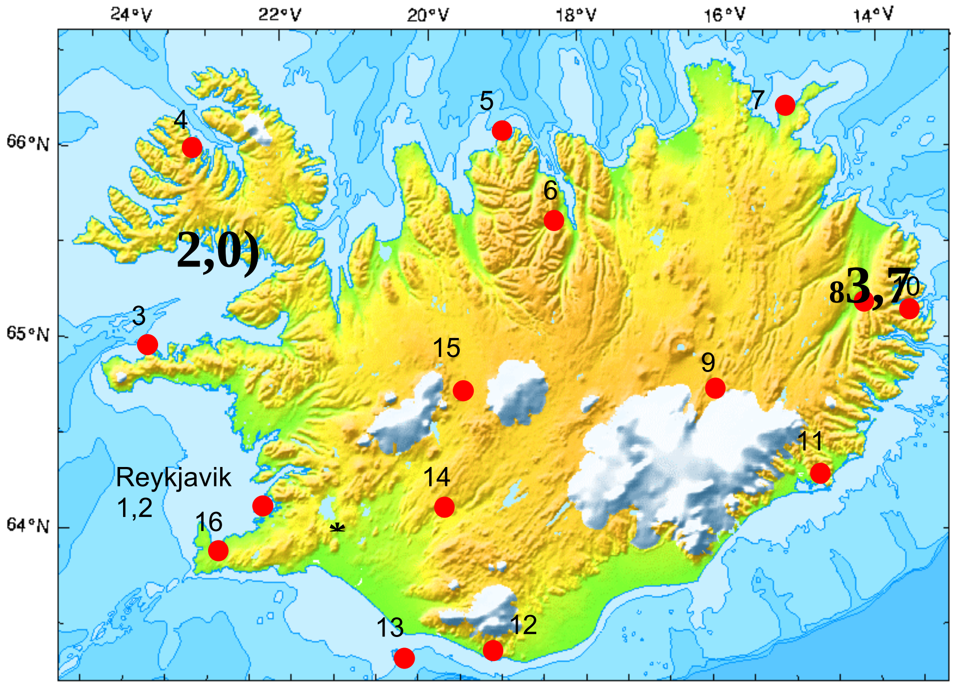


**Figure 1. Map of Iceland showing exposure test locations**

* Irafoss (see text)

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**Table 1. Corrosion rate of carbon steel and zinc after one year's exposure**

| No. | Location | Corrosion rate (µm /first yr.) Carbon steel | Zinc | Climate [1] measured |
|---|---|---|---|---|
| 1. | Reykjavik 1 | 20.7 | 2.3 | 1 |
| 2. | Reykjavik 2 | 18.6 | 2.7 | 2 |
| 3. | Olafsvik | 21.5 | 2.7 | 2 |
| 4. | Bolungarvik | 15.8 | 2.0 | 1 |
| 5. | Siglufjordur | 13.0 | 1.4 | 1 |
| 6. | Akureyri | 5.5 | 1.6 | 1 |
| 7. | Thorshofn | 17.0 | 1.6 | 2 |
| 8. | Egilsstadir | 3.7 | 2.5 | 1 |
| 9. | Kverkfjoll | 1.5 | 1.6 | 3 |
| 10. | Neskaupsstadur | 12.8 | 3.9 | 1 |
| 11. | Hornafjordur | 24.9 | 1.6 | 2 |
| 12. | Vik i Myrdal | 18.7 | 2.3 | 1 |
| 13. | Vestmannaeyjar | 36.1 | 2.3 | 1 |
| 14. | Burfell | 22.3 | 2.6 | 1 |
| 15. | Hveravellir | 4.2 | 2.2 | 1 |
| 16. | Svartsengi | 35.3 | 2.7 | 2 |
| | Average | 16.98 | 2.25 | |
| | Standard deviation | 10.256 | 0.632 | |
| | Variation | 0.604 | 0.281 | |

[1] Climate measured :
1 at weather station
2 at a nearby location ( < 20 km )
3 distance to weather station > 100 km

## 3 THE CLIMATE AND CORROSIVE AGENTS IN ICELAND

The atmospheric corrosion of metals is discussed at length in the literature. The standard ISO 9223 specifies the key factors and Haagenrud (1997) gives a very good overview.

The climate factors used in the project are measured by the Icelandic Metrological Station at, or nearby, most of the test stations. Measurements of temperature, humidity, wind-speed and direction are logged automatically each 1 or 3 hours, and in some instances precipitation is also available. From the start, it was clear that some spot measurements of air pollution would be needed, but it was decided to wait for initial results on corrosion to see where to concentrate the main effort.

For the exposure locations, the climatic factors of importance in this context can be described as follows.

Temperature

The climate is temperate and at the corrosion locations, the yearly average temperature varies locally from 0.6 (location 15) to 6.4 °C (location 12). Temperature is seldom higher than 20 °C, or lower than -20°C.

Moisture

The weather is humid at the coast all around the island, but drier in deep fjords and some places in the highland. At the coast, the average monthly values vary slightly and the yearly average value of relative humidity (% RH) is the interval of 78 - 83 % RH and slightly higher in the highlands.

Time of wetness (TOW)

TOW according to the definition of ISO 9223 (T>0°C, RH>80%) is in the interval 1449 hrs. (location 10) to 6324 hrs. (location 13).

Rain

Climate in southwest is wet, but noticeably drier in the north. The yearly precipitation at the locations is from 860 mm (location 15 ) to 3375 mm (locations 12) .

Wind

The climate is windy all year round. The average yearly wind speed at the locations is measured in the range of 3 - 10 m/s, with one location at each extreme, but usually the value for average wind speed is 4.7 – 7.1 m/s The prevailing wind directions are southeasterly to southwesterly .

$SO_2$

$SO_2$ is considered one of the major corrosive agents of steel and zinc. Iceland has little $SO_2$ pollution, as almost all house heating is done either by geothermal energy, or electricity from hydro power stations. The island is very sparsely populated (2.7 $km^2$/inhabitant), and the average wind speed is high. Thus, problems concerning air pollution are uncommon and only limited measurements are made of these agents. $SO_2$ is only measured regularly in Reykjavik, and the yearly mean, near a heavily travelled road was measured in 1991 as 3.2 $\mu g/m^3$. In general, the measured value in Reykjavik can probably be considered as a maximum value for Iceland, if one excludes local climate near two aluminium factories. Exposure location 16 is located near a geothermal power station, with a known high concentration of corrosive agents ($SO_2$) in the steam.

Salinity

It is known that salinity greatly affects the speed of corrosion, and even in highly populated areas this effect can be more than that from $SO_2$, especially as the content of $SO_2$ is decreasing in many places, as discussed by Haagenrud (1997) and reported in '*Hot Dip Galvanizing*' (2001). The chloride deposited is only measured regularly at two places in Iceland: in the capital Reykjavik, and at Irafoss which is inland from the south coast.

Salinity $Cl^-$ (mg/m2,d)

| | yearly mean | highest monthly mean |
|---|---|---|
| Reykjavik (1985) | 19.8 | 53.0 |
| Irafoss (1985) | 10.0 | 15.2 |

It is known that salinity concentration is highest at the southwest parts of the country and lowest in the northeast parts (south to southwesterly winds prevail in Iceland). Salinity is carried by wind over great distances and in southern storms, saline deposit is observed on structures on the north coast, despite highlands in the middle of the country that, depending on direction of wind from coast to coast, have an average height of 400 - 600 m . Concentration of salinity in air due to wind must depend on many factors, including topography, wind speed and direction, as well as distance from location to sea in line with the wind direction. A study of salinity, which takes into account how salinity transfers from sea to air, and the effect of this on the corrosiveness of the climate, was reported by Cole *et al*. (1999) and wind speed was found to be a very important variable.

## 4 CORROSIVITY CLASSES ISO 9223: 1992

The standard, ISO 9223, classifies the atmosphere according to direct measurement of corrosion rates on one hand, and on the other, the value of climatic parameters (time of wetness, sulfur dioxide and airborne salinity).

Based on measured corrosivity, the corrosion class for steel in Iceland is in the range 2 - 3 and for zinc, 3 - 4. Classification based on information about the climate gives the corrosivity class for both steel and zinc as ranging from 3 to 4 (or even 5). The two methods thus, do not give exactly the same result, as steel corrodes slightly slower than expected from the climatic classification.

## 5 DOSE RESPONSE FUNCTIONS FOR CORROSION SPEED IN ICELAND

The models of the effect of different climatic factors on the corrosion of metals, or dose response functions, easily become complicated, and it is probably most economic to simply measure the effect by mapping corrosivity in the area in question. Scientifically, however, the models are of great interest as an aid in understanding effect of climate on materials and as damage functions.

In the literature, Haagenrud (1997) showed an overview of dose response functions. These functions are based on a different number of climatic variables and the composition of them is somewhat different, and so is even the number of variables. In most of the functions, (in fact all, except one) the $SO_2$ plays a major role, though humidity, time of wetness (TOW), sulphur (S) and clorides (Cl-) are also used as variables. The correlation coefficient (multiple linear correlation) found for the different models varies between 0.67 and 0.94.

When the dose response functions were tested on the data from our project, with different values for the unknown factors (S , $SO_2$, H+, Cl- etc.) based on an intelligent guess, it was apparent that the functions are poor estimators for corrosion speed in Iceland. The reason for this is probably that the $SO_2$ content is usually low, and thus, not a main factor in the corrosion. On the other hand, salinity is high at times. The measured corrosion rate of steel is much higher at the seashore, than at locations inland, but the corrosion of zinc is not as dependant on location. For the first years results therefore only the corrosion of steel will be modeled in dose response funtions.

For corrosion of steel, the airborne salinity and moisture, or time of wetness, are probably the main factors. Salinity comes from the sea, and most of the test sites are at the seashore. Lacking measurements of salinity, these can probably be estimated by the wind speed and distance from the shore. In the multiple regression models, the effect of salinity is therefore estimated by two methods;

– the average wind speed in directions from the sea divided by distance from shore

– the average wind speed from sea, multiplied by the fraction of time these wind directions prevail.

Different multiple regression models were tested on the data, but the two shown gave best results:

Model 1 : Corr (μm/first yr) = $23.031\,(w_{sa}/L)^{0.2627}\,(TOW)^{0.4704}$

Model 2 : Corr (μm/first yr) = $10.055 + 3.942\,(TOW) + 4.539\,(TOW)(w_{sa}/L)$

where $w_{sa}$ average wind speed (m/s) from direction of sea

L distance from shore (km)

TOW time of wetness in accordance with ISO 9223

(T>0°C, RH>80%), as a fraction of total time

| | Standard error of estimate | Correlation coefficient R |
|---|---|---|
| Model 1 | 6.02 | 0.829 |
| Model 2 | 6.28 | 0.812 |

Model 1 has certain similarities with the model shown by Cole *et al.* (1999), model 2, on the other hand, is of the more traditional type used by many, as revised by Haagenrud (1997). It is interesting to note that two so different models show similar quality of fit to the data, more datapoints (higher number of test locations) would be of of help to decide the appropriate damage function.

Taking the frequency of wind directions from sea into account did not improve the models, on the contrary, the fit became considerably worse. The calculated corrosion rate, in accordance with model 1, is compared with the measured rate in Figure 2. As can be seen from the figure and the values for standard error and the regression coefficient, the regression model fits the measured results quite well. Both models improved considerably, if location 14 was excluded, but this is considered to depend on local reasons as explained next page and not that the point is an outlier of some kind.

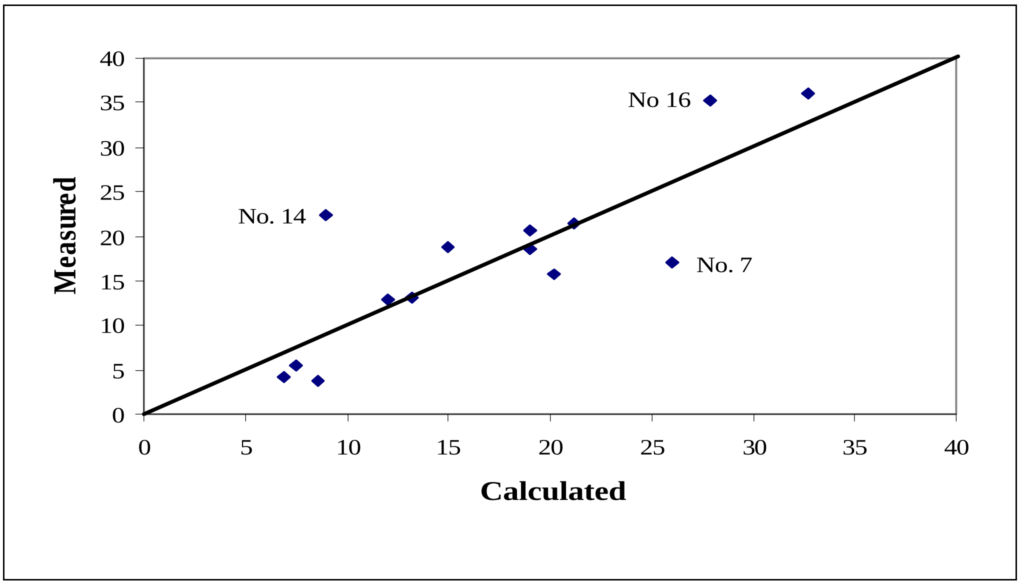


**Figure 2. Corrosion of weathering steel (μm/first yr.)-Model 1**
comparison between calculated and measured values

## 6 DISCUSSION

Even these initial, first-year results give information about the atmospheric corrosion in Iceland which makes comparison with other countries much easier. The corrosivity of the atmosphere is in classes 2 - 4 (ISO 9223). Dose response functions in the literature do not seem to be appropriate for Icelandic conditions, which shows the impact of corrosive agents in different countries can be very different. Airborne salinity seems to be a very important factor in corrosion of steel in Iceland. The measured corrosivity shows great variability and it is evident that this can be explained in different ways. The atmospheric conditions vary greatly between the test locations and the measured samples are few. For instance, results are not sufficiently evenly spread to positively determine if the two highest returns are part of the distribution, or are exceptions (outliers). However, these two points are not the results showing the greatest errors in the regression models, and can probably anyway be explained by differences in climate.

The regression models show a poor fit for the thre points no. 7, 14 and 16. For exposure site no. 7 the calculated value is higher than the measured value. The location is unsheltered on the north-east coast and the model probably overestimates the effect of wind from the sea. The exposure site no. 14 is on the other hand about 50 km inland and the calculated corrosion is lower than the actual one. The surface of the area from shore to the site is very even and without any hills, the model underestimates the effect of wind from sea in this topography and at this distance. Number 16 is underestimated in the calculations. The exposure site is located near a geothermal power station, where it is known that the steam is very corrosive. How and whether this affects the testing site also, is not clear at the moment.

The two sides of the sample are not entirely in the same situation, one being more sheltered than the other. The corrosion is measured as weight loss of the sample, and then calculated as weight loss (or reduced thickness) on unit area, based on the size of the sample and taking into account both sides. But, it is not certain the samples corrode evenly on both sides, at all locations (closed tube specimens are a better solution). The effect of rain washing the samples clean, or of salinity deposited only on the outward (upper) side of the sample, must have different effect on the corrosion, depending on frequency of rainy days, etc. On the whole, the timing effect for different series of agents (time series), at different locations, can play a major role in the corrosivity of the atmosphere.

## 7 CONCLUSIONS

Corrosion of metals, especially steel, in Iceland varies videly for different locations, with the main corrosive factor seemingly being airborne salinity. The corrosion rate has only been measured over one year, but the project will continue for a total of ten years. The content of salinity will be measured at some places, to obtain knowledge about what values to expect, and concentration of $SO_2$ will be measured at location 16 and one other for comparison. These measurements will probably be made periodically, and over limited time. Further data processing of the effect of climate on corrosivity of metals in Iceland will continue when the results after three year's weathering have been gathered. In the continuation of the project, the question of different corrosion on the two sides of the sample will be examined. The time series for climatic factors of different locations will also be studied.

The first samples for assessment of painted surfaces will be demounted at the 3-year timepoint but some of the samples already show some faults, assumed to be mainly due to sandstorms.

## 8 ACKNOWLEDGEMENT

The project is conducted in cooperation between The Icelandic Building Research Institute, Pétur Stefánsson - Almenna Verkfræðistofan and Pétur Sigurðsson - Vekfræðiþjónusta, and with financial support from the following Icelandic entities: The Icelandic Research Council, Housing Financing Fund, Avalanche Fund, National Power Company, Public Road Administration, Westfjord Power Company and a number of smaller private companies. The authors wish to thank all parties for their help in making the project possible.

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# Extending Service Life Of Buildings And Building Components Through Re-Use

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**Summary: From an environmental point of view, the extension of the service life of a building is only useful under the condition that the environmental load generated by the maintenance and renovation activities, and the use during the remaining lifetime of resources like water and energy, is less than the load generated by demolition, sustainable new construction and use.**

**Resource conservation in this context means a very important minimisation of the use of resources, making use of renewable resources and of the ideas of biotecture for example. The combination with the re-use of buildings parts, components and materials (bcm's) is important in this respect. These themes are further elaborated in this paper.**

**LCA-based assessment methods have to show the optimum selection of design approaches.**

**The paper concludes that there still are significant problems to be overcome when applying these assessment methods, as a result of a lack of technical data and missing links in the methods themselves.**

**Keywords: Renovation, re-use, sustainability, waste reduction**

## 1 INTRODUCTION

Building and construction is one of the engines of the economy, considering their contribution to the Gross Domestic Product, Gross Fixed Capital Formation, and employment. The general expectation is that there will be economic growth in the future too. Due to the strong relation between building activities and economic development, there is an increased need for more and bigger housing, and utility buildings (Erkelens, 1991, p. 25).

As building has a significant impact on the environment it is justified to look at means to reduce this impact and to seek for sustainability. The second-best option, after not building at all, is to introduce the concept of extreme sustainability within building, which is far beyond the current building practice.

A building is in fact a structure that protects the users against all sorts of external influences and provides a comfortable space for all human activities. Like man, a building has a life period, which starts during the construction, continues during its use, its maintenance phase(s) and rehabilitation phase(s) and ends with its demolition. For all these phases (except the latter one) resources are needed.

A utopian approach for sustainable building would be to develop building products and buildings that do not produce waste, cost energy, require transport etc. and are cheap. Although this will not be elaborated on further, the idea is very interesting.

Resource conservation means that in the whole life span of the building, the use of resources (building materials, energy, etc.) should be such that the environmental impact is as low as possible and / or, better still, zero. Ideally, the combination of resources should have zero impact on the environment or even improve that environment, including its best quality and health. So far this is not yet feasible. A more realistic approach can be sketched as follows:

- Minimisation of the use of resources, and where necessary
- Use of renewable resources applying the ideas of biotecture, as well as
- The extension of service life through re-use and recycling (at all levels from whole buildings down to materials).

Environmental impact assessments are needed to check for the best combination of options, as it is not always the solution with the minimum use of resources that results in the minimum environmental impact. The following sections elaborate on these themes in more detail and discuss the constraints to be faced.

## 2 MINIMISATION OF THE USE OF RESOURCES

Minimisation of the use of resources also implies minimisation of the use of so-called endless resources because they too need energy for transport, exploration, exploitation and manufacturing.

This minimisation can primarily be achieved through reduction of the demand in various ways.

In essence, the reduction (of the amount) of building and construction waste causes reduced demand. The Dutch Foundation for Building Research (SBR 1998, pp. 5,6) reports a reduction of 41% in waste production in a housing project through good management.

Moreover, through the design of compact buildings, the amount of resources (i.e. building materials) can be minimised. This may not be the optimum solution given the local circumstances. Another approach is dematerialization. This means the reduction of quantities of materials needed to serve economic functions. To achieve sustainability von Weiszäcker *et al.* (1997) suggests that the materials input per service unit must be reduced by a factor of 10! This is an interesting thought for building too, but it requires extensive research. A parallel approach may be the development of smart building technology: fewer materials with better performance.

## 3 USE OF RENEWABLE RESOURCES, BIOTECTURE

It is often said that use should be made of renewable resources like wood, flax, hemp for building materials, components and services (like water and forms of energy). These resources have a short regeneration time: the resources can be grown and harvested. However, they have a limited capacity for renewal and can thus be subject to overexploitation. So, there is also a need to cascade these renewable resources. Cascading means the sequential exploitation of the remaining full potential of a resource during its use.

As renewable resources are not always environmentally friendly (e.g. the need for fertilizer to make materials grow), LCA's need to be made for them as well. At times this may lead to the conclusion that another (non-renewable) resource is a better option.

Another suggestion is to utilize grown structures and living plants, and space enclosed by trees. This so-called biotecture (Fraanje, C3-4.2) can be applied in the design of buildings. This has important resource-saving potentials and can contribute to a more natural regulation of the internal condition.

## 4 EXTENSION OF SERVICE LIFE

The extension of the service life of buildings and building components, products and materials is the third approach to be reviewed here.

### 4.1 Building

The ability to extend the service life of a building or parts of it depends on different factors.

Schulte (1997, p. 37) distinguishes three groups of aspects that are decisive for any decision on the future of an existing building:

1. Societal aspects: socio-economic, historical, juridical, financial and ideal aspects;
2. Urban aspects: historical, morphological, spatial / functional and use aspects
3. Building technological aspects: infrastructural, structural, manufacturing and building physical aspects.

An important question is whether society allows for the extension of service life: is it still the "fashion" to have such a building. Does it fit into the contemporary vision / policy.

Another point can be that the building, although old, contributes significantly to the 'value' of a location; an artistic value making it worth extending its service life, or a building with a historical value.

More complicated is a building that functions according to the original set-up, but does not fit into the functional / technical comfort requirements as formulated in the contemporary building regulations.

In this case it depends on the remaining qualities of the building whether the service life of either the building as a whole will be extended or only parts thereof, or whether the service life of just the structural framework of the building will be extended while the rest is being replaced or renovated, see 'Fig. 1'.

**Fig. 1 A fully stripped building for re-use**

### 4.2 Building parts, components and materials (bcm's)

The service life of building parts (façade, roof, etc.) components (windows, panels) and materials (glass, timber, tiles) also depend on the above mentioned aspects. Through renovation and rehabilitation activities bcm's will be sometimes replaced, repaired or left untouched, removed or demolished, partly or as a whole.

The service life of bcm's can also be extended through reuse in another project (e.g. the Carrousel experiment in The Hague whereby cleaned and repaired bcm's are stored for future reuse in renovation projects with a limited remaining life span), although this is more complex. The option of another location seldom is used for the main structure of a building.

From the point of view of the optimum use of resources, the best option would be if the service life of bcm's could be extended in this same building. In addition, we can consider recycling, although this option has a minor preference. Ultimately, discarding should be prevented.

### 4.3 Design requirements

Whether the service life of a building, a building part, a component or a material can be extended, also depends on the following:

1. The design proposals through which different quantities of bcm's have to be removed.
2. The initial design phase in which the future reuse of bcm's is already taken into account.
3. The quality of the bcm's that indicates whether they are reusable in this (or another) project. (This depends on the quality of the used materials, details, craftsmanship, maintenance, way of use and the method of demolition or demounting).

### 4.4 Environmental requirements

The consequences of the various options for the extension of service life have to be checked with the LCA programs, which take into account the use of materials resources, energy, waste and emissions. This results in an ordering of the design proposals along increasing total environmental impact. Usually, in daily practice, the economic costs of the proposals are decisive for the final selection. The initial costs in particular are focused on. A form of subsidy could stimulate a more environmentally friendly choice.

## 5 RESEARCH

Apart from saving resources by reusing a building, it is very useful during renovation to look into the reduction of the waste production. If renovation projects could be developed with a minimum waste output, this could contribute significantly to sustainable construction (Erkelens, 2000, C3-2.7). This involves anticipative thinking about the intended demolition: Can it be kept to a minimum, can the materials and components be taken off without damage and can they be re-used in the same project preferably for the same purpose? Or, if that is not the case, can they be re-used in other projects?

For new construction this means anticipating future re-use, through the application of other than (e.g. gluing) jointing techniques, design for deconstruction, and the use of (industrial) flexible and demountable components. Furthermore, the development of special demolition methods is of importance. As a start, the Technische Universiteit Eindhoven has developed a model to investigate the flows of all the building parts, components and materials (bcm's) in a renovation project, see 'Fig. 2'.

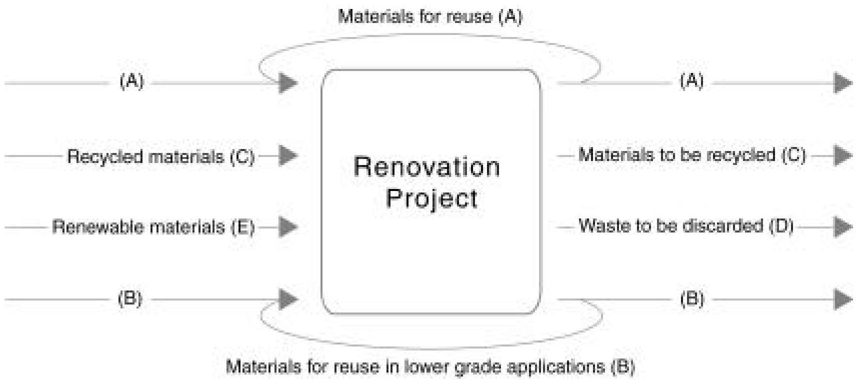


**Fig. 2. Minimum waste research model**

The model shows bcm's on the output side. Those of flow (A) are re-used for the same application after some repairs. Those of (B) can be re-used for lower grade applications. Those of (C) can be recycled and those of (D) can be discarded as waste. On the input side we see bcm's of the type of flows (A) and (B), and flow (C) consisting of recycled bcm's and flow (E) with new but renewable bcm's.

While researching a building project one first needs to establish the 'remaining' potentials of the existing bcm's in terms of quantity, quality, remaining life span, ease of dismounting etc. Furthermore, the potential reuse has to be sorted out and its level determined (primary reuse, secondary reuse, recycling or discarding).

The various options for part renovations will have different environmental impacts. The variation can be in floor plans, materials (re-)use etc. The various designs for a total renovation plan gives an order of increasing environmental impact. This guides us to find the best design proposal with a reduced environmental impact. Alternatively we can tune a preferred design in such a way that the impact is reduced.

## 6 CASE STUDY

A complex of 248 houses in the suburb Lievendaal of Eindhoven (the Netherlands), was used as a case study. These houses were built in 1949 and partly renovated in 1977. The coming renovation is planned for 2002. The total estimated lifetime is up to 2026. Kevin de Rond and Dieter van Riel (MSc. thesists) made a number of calculations based on three scenarios: (1) A continued use without changes, maintenance only; (2) a renovation in 2002; and (3) demolition and new construction in 2002 lasting up to 2026. The following 'Table 1' indicates the changes in the building parts over time. Some parts can be used continuously while some others have to be replaced.

**Table 1. Lifetime of various building parts**

<table>
<tr><th>1949</th><th>1977</th><th>2001</th><th>2026</th></tr>
<tr><td colspan="3">Foundation, facades, floors, wall plates, purlins</td><td></td></tr>
<tr><td>Windows and -frames<br>Internal door/frames<br>Ceiling plates</td><td colspan="2">Windows and -frames<br>Internal door/frames<br>Ceiling plates</td><td></td></tr>
<tr><td colspan="2">Roofing-plates, tiles, rafters<br>External finishing of dormer window<br>Roof gutters, rain-water pipes</td><td>Roofing plates, tiles, rafters<br>External finishing of dormer window<br>Roof gutters, rain-water pipes</td><td></td></tr>
<tr><td>Internal walls<br>Shower, toilet, kitchen,<br>wall tiles<br>Mains, installations</td><td>Internal walls<br>Shower, toilet, kitchen,<br>wall tiles<br>Mains, installations</td><td>Internal walls<br>Shower, toilet, kitchen,<br>wall tiles<br>Mains, installations</td><td></td></tr>
</table>

Based on detailed LCA calculations with Simapro and Ecoquantum, estimates were made of the total impact for the scenarios. Both Simapro and Ecoquantum are dutch LCA computer programs which are interrelated. As these LCA calculations are not yet totally optimised, the students had to sometimes estimate the input. Calculations were made for energy, waste, emissions and use of resources.

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On the condition that the maintenance scenario is set to 100, the environmental impact scores of the scenarios are as follows: 157 negative points for scenario 2 and 274 negative points for scenario 3. The main relative environmental impacts in scenario 2 compared to scenario 1 are the roof and the installations.

The follow-up of the project will consist of an improved renovation design for scenario 2, which reduces the score by 26 points, compared to the original renovation in scenario 2. This research project is on-going and during the conference in Brisbane some more of the results will be presented.

## 7 MISSING LINKS / CONCLUSIONS

The case study showed a number of specific problems that a worth reporting.

The reality shows specific problems because of missing or incomplete as-built information and a lack of proper calculation tools.

### 7.1 Environmental gaps

Calculation programs are not yet completely satisfactory. For proper information one depends on the industries for building materials and components**.** For example, it is difficult to obtain calculations for old constructions and materials. Furthermore, it is difficult to establish which life span has to be applied when making comparisons for existing buildings, reused components or new materials. Besides that, it has proved problematic for LCA's to add up environmental aspects such as energy, waste, raw materials and emissions.

### 7.2 Technical gaps

Up until now designers have lacked complete insight into options for design variations. It makes a great difference if, for example, a wall is placed in a plan or moved just 10 cm more to one side.

Moreover, there are no simple and clean methods for demolishing or demounting. This would make options for reuse in the same project much more easy to come by.

### 7.3 Economic gaps

Naturally, economics are seldom good for the environment as a number of hidden costs are not calculated or taken into account. Through incentives like subsidies for reuse, levies for discarding and a premium for the return of good bcm's, the most economic solution may be different from the traditional ones.

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# Failure Mode Effect And Criticality Analysis For Risk Analysis (Design) And Maintenance Planning (Exploitation)

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**Summary: The ISO 15686 (ISO, 1998) set of standards "Service life planning" is developing an integrated design framework for buildings and constructed assets, incorporating technical, economic and environmental aspects.**

**In this context, CSTB has developed a "risk analysis" and "maintenance planning" tool, based on the use of the Failure Modes and Effects Analysis (FMEA). It includes:**

**first, a structural analysis which allows us to identify morphology, topology and physico-chemical constitution of the product and its components,**

**second, a functional analysis leading to the identification of the various functions ensured by the product and its elements, as well as a description of its environment,**

**These first two steps lead to the behaviour modelling of the product.**

**finally, an FMEA in order to identify failure modes (exhaustive search for the behaviours, degradations and failures of elements), their causes and effects, taking into account the potential problems and errors which could occur during the process.**

**This step leads to the identification of degradation and failure. We thus can build the failure modes (or failure scenarios).**

**The methodology proposed simplifies each step of the approach, and provides us with a graphical representation of the product behaviour.**

**Keywords. Durability, FMEA, failure modes, risk analysis, maintenance**

## 1 INTRODUCTION

### 1.1 Context

Non-quality or poor quality problems in the building domain (representing 10% of the French building turnover, i.e. 6.5 to 7.5 billion Euros according to Le Brigand (1998)), the part of maintenance and operating stages in the cost of a building (60 to 70% of the overall cost (Perret 1995)), as well as the new aim "Sustainable Development[1]" (natural resources preservation, energy saving, etc), led us to work out methods and tools for durability assessment.

[1] "A development which answers the needs of the present generation without compromising the ability of future generations to answer theirs" (Charlot-Valdieu and al 1999).

### 1.2 Objective

In this paper, we present the durability assessment methodology (with reference to (Lair 1999)), and then propose additional developments and applications to the sketched methodology.

In particular, we include the tool in the quality approach, during the design stage (risk analysis – help for design), as well as for the management of existing buildings (maintenance planning – diagnosis / maintenance / repair procedures).

## 2 METHODOLOGY

### 2.1 General

FMEA has been used since the sixties in the aeronautics and car industry and is thus an efficient tool for the safety and risk analysis of these systems (Leroy 1992). It is intended to check the ability of a system and its parts to meet user's needs and either is generally used during the design stage in order to target and examine weak points, before mass production (Modarres 1993). It allows us to identify the potential future behaviours (success or failure) of construction products.

It leads to an exhaustive list of all failure modes, i.e. all problems that might occur:

- at each stage in the construction process (design, manufacturing, installation, transportation, storage, etc),
- in service (influence on the product's performance, degradation or failure of one of the product's constituent materials/components constitutive material/element).

In the following paragraphs, each step in the methodology will be presented and illustrated with basic examples.

### 2.2 Structural analysis

#### 2.2.1 Principle (Figure 1)

The principle of this study is to built an accurate description of the product:

- by dividing it into elements,
- and by identifying the constituent materials.

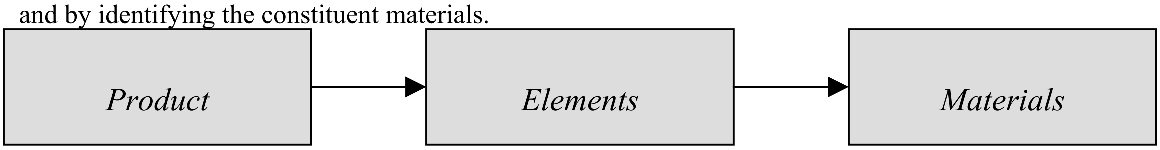


**Figure 1: Structural analysis**

This first analysis allows us to describe the structure of the product being studied. The following are identified (Baloche and al. 1999):

- morphology (geometrical shape, dimensions, etc),
- topology of relations with other objects,
- physico-chemical composition of its constituent elements and their own description.

#### 2.2.2 Example: Roofing system

The following graph is a representation of a flat roof system (left part of Figure 2), and its corresponding structural diagram (right part of Figure 2).

Outdoor and indoor environments have to be defined. They are composed of all potential climatic and use factors that correspond to the environment in which the product will be in service (Figure 3).

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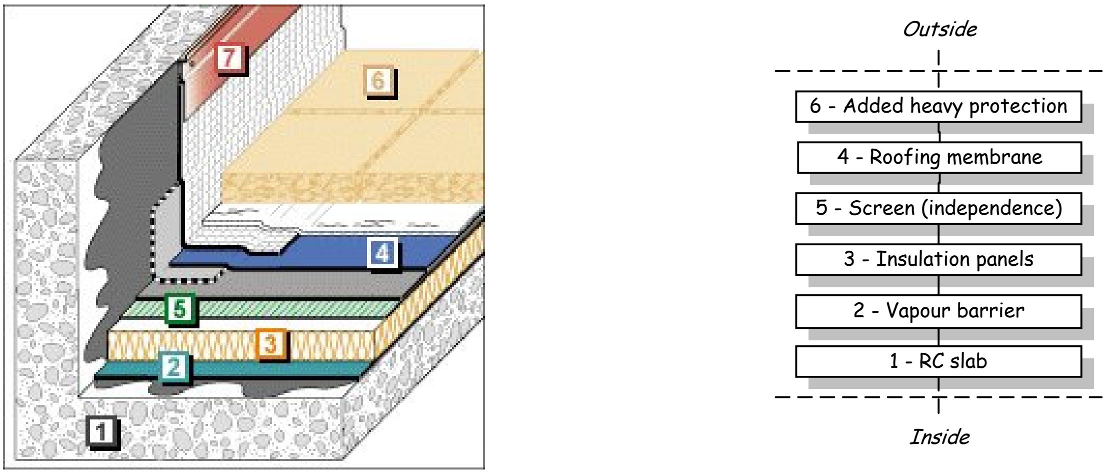


**Figure 2: Section of a flat roof system and structural representation**

| Outdoor | 2.3 Indoor |
|---|---|
| Temperature (high or low) | Temperature |
| Rain, snow, hail, ice | Vapour |
| Sun | Indoor pollutants |
| Wind | |
| Pollutants, weed-killer | |
| Vegetation, moss, lichen, etc | |
| Shocks and loads (persons) | |

**Figure 3: Climatic and use factors**

A list of common climatic factors is defined in order to help the user (generic factors). Some specific factors have to be defined according to the use (chemical products in some industries, etc).

## 2.4 Functional analysis

### 2.4.1 Principle

A product meets users' needs. The functional analysis of the needs expresses the needs in terms of functions.

We also have to identify the functions of each element (i.e. the role of each element in the overall behaviour of the product).

### 2.4.2 Example

As an example, a flat roof system should ensure two major functions:

- thermal insulation (against temperature gradient),
- watertightness (role of roofing membrane against rain, snow and hail).

In addition to these functions (user's needs), we have to consider the function of each element:

- the independence screen (element number 5) of a roofing system limits the strains in the roofing membrane, due to the movements of insulation panels (especially thermal dilatation),
- resistance of all elements to the environment (temperature, sun, rain, etc).

## 2.5 Behaviour analysis

### 2.5.1 Modelling principle

A product is defined as an order of elements and materials that ensure a set of functions.

In the building domain, “The product fulfils a function” could be expressed as “The building product transforms climatic factors (between input and output)”. It acts as a filter between two environments, filtering heat flows between outdoor and indoor environments (thermal insulation), stopping water from outdoor (watertightness of a roofing system), etc.

But, the same climatic factors can have an impact on its constituent elements and could involve: modification of the materials properties, degradation and even failure, etc

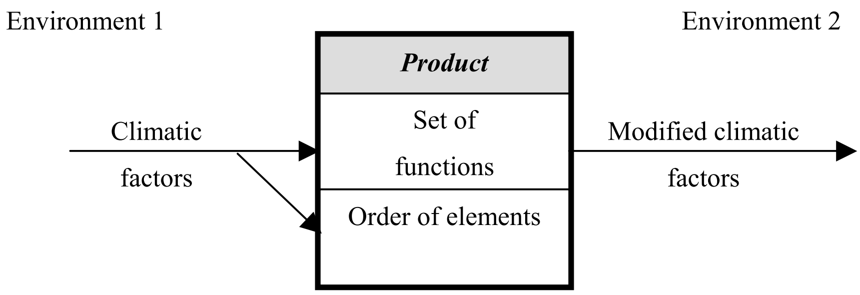


**Figure 4: Product representation**

We can express the behaviour of elements on the same diagram, showing on the one hand that they fulfil a function, on the other hand that they are likely to be modified and degraded by climatic and use factors.

### 2.5.2 Behavioural graph

In this context, once we have identified the structure and the functions of the product, the next step consists in modelling its behaviour.

The approach adopted is opposite to the structural analysis: from the behaviour of the materials, we deduce the behaviour of the elements and then the behaviour of the product itself (Figure 5).

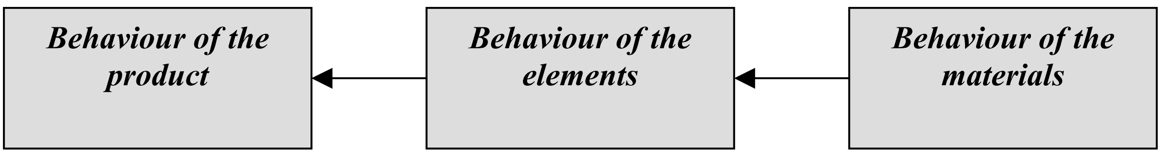


**Figure 5: Behaviour analysis**

For that purpose, a behavioural graph is plotted. The product is composed of n elements, placed “in series” or “in parallel”.

- each block being an element,
- and each link being a relation between blocks (physical contact, flow transfer, etc).

Figure 6 is an example of a very simple graph (only series relations). It ensures a barrier role between outdoor and indoor environment, as a composite wall:

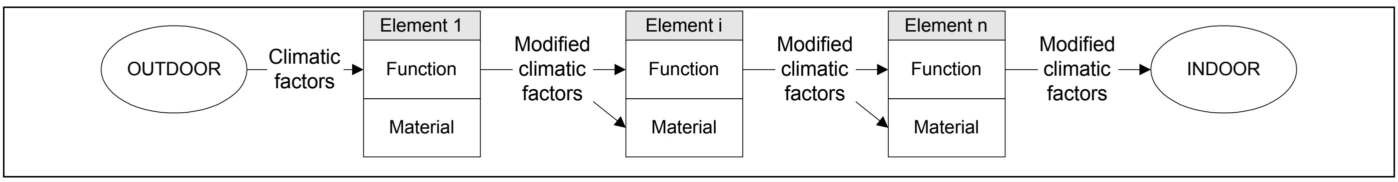


**Figure 6: Product behaviour**

### 2.5.3 Special cases

In order to have an accurate representation of the product, additional information is required:

- cavity (as the cavity between the internal and external glass of a double glazing unit or the cavity in a pipe),
- interface, either simple contact (added heavy protection on roofing membrane) or fixed interface (as the interface between seal and glazing).

They are both represented as elements:

- in the first case to take into account thermal and acoustic characteristics, as well as flows (air, water, etc) and deposit (condensation, dust, etc),

- in the second case, mainly to take into account problems of compatibility/incompatibility of materials, gluing properties, watertightness and airtightness.

### 2.5.4 Nominal behaviour (t = 0)

From this graph, we can deduce the nominal behaviour of the product, i.e. we know the response of a product (and its element) to a given set of climatic and use factors.

At t = 0 (without any degradation and considering that the product was correctly installed/implemented), we can identify the initial state of stresses. It is summarised in the following table:

- outdoor factors go from the top to the bottom,
- indoor factors go from the bottom to the top.

| | OUTDOOR / INDOOR | High or low Temp. | Rain | Snow | Hail | Ice | Sun (UV) | Wind | Pollut., weed- | Vegetation (roots) | Moss, lichen, … | Shocks | Loads | Temperature | Vapour | Indoor pollutants |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6 | **Added protection** | x | x | x | x | x | x | x | x | x | x | x | x | | | |
| 4 | **Roofing membrane** | x | x | x | x | x | | | x | x | x | | x | | | |
| 5 | **Independ. screen** | x | | | | | | | | | | | x | | | |
| 3 | **Insulation panels** | x | | | | | | | | | | | x | x | | |
| 2 | **Vapour barrier** | | | | | | | | | | | | x | x | x | x |
| 1 | **RC slab** | | | | | | | | | | | | x | x | x | x |

**Figure 7: Environmental stresses – Initial conditions**

We thus have information on:

- the various flows in the product (e.g. thermal flow),
- the key elements (protecting elements: role of heavy added protection for roofing membrane against UV, shocks, etc),

## 2.6 Degradation and failure analysis – Failure modes (t > 0)

Given the "nominal" functioning of the product, we start working on degradation and failure study. We study the behaviour of the product if the behaviour of a material or element deviates from its normal behaviour.

The principle is an iterative study.

Step 1: Degradation of elements due to climatic or use factors

We first analyse the influence of initial environmental stresses on each element.

For instance, we have to study the behaviour of the roofing membrane towards high and low temperatures, water (rain, snow, hail, etc), ice, various pollutants, vegetation (including moss, lichens, etc) and shocks.

The methodology used allows us to take into account not only these "single" factors but also the combination of these factors:

- combined factors from the same environment, either concomitant factors (water and low temperature from outdoor environment → freezing/thawing cycles) or successive factors (sun and high temperature followed by rain → thermal shock),

- combined factors from different environments (high temperature from outdoor and low temperature from indoor → temperature gradient in the element).

Knowing the element (and its constituent material) as well as the potential stress factors, we identify the potential behaviours:

- wind can result aggregates being blown away (from heavy added protection),
- water on roofing membrane can cause the removal of solvents,
- high temperatures can involve thermal ageing (hardening),
- low temperatures can involve embrittling, and make tear easier under load stresses.

Step 2: Structural or environmental modifications (degradation or failure identification)

The behaviour or the degradation of the first step leads to the modification of the environment or the structure.
As examples to degradation and failure, the following was observed:

- under temperature stress, the roofing membrane gets old. This ageing causes hardening. Although roofing membrane is degraded, it still fulfils its main function: watertightness, moreover it will not be able to withstand mechanical stresses as well as initially.
- if the heavy added protection has blown away from some parts of the roofing system (failure of the "protection of the roofing membrane" function), this roofing membrane is no more protected against UV and temperature. This comes down to a new cross in Figure 7 (column Sun(UV), line Roofing membrane). The initial state is updated and becomes a "State 2" condition for which we have to study the effect of UV on roofing membrane.

Step 3: Degradation of elements due to updated climatic or use factors

Given the modification of structural or environmental conditions (step 2), we now study again the behaviour under new environmental conditions (action of environmental factors on elements due to structural modifications, mainly loss of protection):

- photochemical ageing of roofing membrane due to the UV stresses,

Again we identify the modification of both structure and environment (step 2), then step 3, and so on…

For instance, once the heavy added protection has failed since it was not thick enough to protect the roofing membrane, the latter is stressed by UV and temperature (step 2). It will fail by cracking or tear, and then will not fulfil its "watertightness" function any more. Rain, pollutants, … will go through the roofing membrane and reach the insulation panels.

OUTDOOR

| | | High or low | Rain | Snow | Hail | Ice | Sun (UV) | Wind | Pollut., weed- | Vegetation | Moss, lichen, etc | Shocks | Loads | Temperature | Vapour | Indoor pollutants |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6 | Added protection | x | x | x | x | x | x | x | x | x | x | x | x | | | |
| 4 | Roofing membrane | x | x | x | x | x | x | | x | x | x | | x | | | |
| 5 | Independ. screen | x | | | | | | | | | | | x | | | |
| 3 | Insulation panels | x | | | | | | | | | | | x | x | | |
| 2 | Vapour barrier | | | | | | | | | | | | x | x | x | x |
| 1 | RC slab | | | | | | | | | | | | x | x | x | x |

INDOOR

**Figure 8: Environmental stresses – State 2**

Note: We also take into account construction process problems, i.e. problems occurring during the following stages: design, transportation, storage, installation, use, etc.

We use the 5M rule to identify the various potential defaults, negligence or errors concerning:

- Material (quality, incompatibility, surface quality, etc)
- Manpower (surface cleanness, dimensions, etc),
- Middle, i.e. environment (temperature, humidity, etc),
- Means (non-adapted tools, etc),
- Method (time limit, etc).

They intervene as structural or environmental modifications in step 2.

The failure of a reinforced concrete element could be due not only to the impact of environmental factors impact ($CO_2$, …) but also to vibration, temperature, curing, position of reinforced bars, etc.

# 3 RESULTS AND LIMITS

## 3.1 Results

We thus obtain:

- information about the "Nominal behaviour" of the product in a given environment in order to document "reference service life" (Sjöström and al., 2001),
- information on the degradation and failures, expressed as a list of degradation and failures (FMEA table) or a failure tree (with scenarios).

In this table are listed for each element, the modes, causes and effects.

In column "Causes", we distinguish errors and problems occurring during the process (cause type 0), and the different levels of degradation or failure: cause type 1 (initial environmental stresses), cause type 2 (occurring after the degradation or failure of an element), and so on …

| Function | Element | Modes | Causes | Direct effects | Indirect effects |
|---|---|---|---|---|---|
| Watertightness (outdoor humidity) | Roofing membrane | Cracking, tear | 1 -Load and wear | Cracked membrane | Water penetration (to insulation) |
| | | | 1 - Dimensional variation of support (bending) | | |
| | | | 1 - Adherent ice (compressive stresses) | | |
| | | Local lifting | 1 – Wind (air flow) | | |
| | | | 1 - Vapour pressure | | |
| | | Slipping | 1 - Temperature - Bitumen slipping | Loss of watertightness performance | Water penetration (to insulation) |
| | | | 1 - Temperature - Excessive quantity of bitumen | | |
| | | | 1 - Overload (slope and friction) | Loss of continuity | Stress on edge |
| | | Perforation | 0 - Punctual load (installation) | Pierced membrane | Water penetration (to insulation) |
| | | | 0 - Punctual load (maintenance) | | |
| | | | 0 - Punctual load (use) | | |
| | | | 1 - Vegetation | | |
| | Vapour membrane (if roofing membrane fails) | Cracking, tear or perforation | Refer to the item Watertightness (indoor humidity) | | |
| Watertightness (Indoor humidity) | Vapour membrane | Perforation | 0 - Problem during installation (concrete surface finishing) | Loss of watertightness performance | Water penetration (to insulation) |
| | | | 1 - Dimensional variations of support (RC slab) | | |
| | | | 1 - Dimensional variations of insulation panels (swelling) | | |
| | | Cracking, tear | 0 – Problem during installation (concrete surface finishing) | Loss of watertightness performance | Water penetration (to insulation) |
| | | | 1 – Dimensional variations of support (RC slab) | | |
| | | | 1 – Dimensional variations of insulation panels (swelling) | | |
| Thermal insulation | Insulation panels | Water absorption | 2 – Roofing membrane failure | Loss of thermal characteristics | |
| | | | 2 – Vapour barrier failure | Swelling | Stress of roofing membrane |
| … | … | … | … | … | … |

**Figure 9: FMEA table of a roofing system (Extract)**

## 3.2 Limits

At the moment, the main limit of the methodology is at this moment the binary reasoning, i.e.:

- we do not have any indication of time before degradation or failure, i.e. progressive degradation (just consider that they will occur anyway),
- we do not "measure" the intensity of the phenomena (just consider that they will occur anyway),

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- we do not take into account geometrical aspect as partial degradation (10% of the surface is degraded for example).

This limit leads to an exhaustive list of degradation and failures, with obviously some improbable and unrealistic failure modes.

Two solutions are currently under study in order to refine the results (select the most relevant modes in terms of frequency or gravity and delete the others):

- developing an expert method for criticality analysis (Probability x Gravity of effects),
- introducing qualitative reasoning in order to take into account progressive degradation.

## 4 INTERESTS AND PERSPECTIVES

This tool could be used at the different stages in the product life:

- Design and installation (risk analysis and maintenance planning),
- Use (maintenance planning or failure diagnosis).

The following table (Figure 10) gives the results and the objectives of this tool.

| **Construction process stage** | | **Results** | **Objectives** |
|---|---|---|---|
| **Design**<br>**Installation** | **1 - Risk analysis** | Identification of weak points of the product (characterised by a level of criticality) | *Improve quality and reliability from design stage by*:<br>- identifying problems,<br>- giving priorities. |
| | **2 - Quality management** | Identification of critical operations (Installation, use) or critical elements leading frequently to degradation and failure. | *Improve the construction procedures (transport, storage, setting up) and use by:*<br>- identifying problems,<br>- giving a check-list of key steps (scale of priority). |
| **Use** | **3 - Preventive maintenance** | - Forecasting of potential behaviours in time.<br>- Assessment of the criticality of possible consequences. | *Improve **proactive** maintenance procedures by:*<br>- identifying problems,<br>- giving priorities,<br>- proposing solutions to maintenance. |
| | **4 – Conditional maintenance** | - Identification of symptoms, warning signs of failure (condition assessment, diagnosis of degradation)<br>- Forecasting of future behaviours (given actual state).<br>- Assessment of the criticality of possible consequences. | *Improve **reactive** maintenance procedures by:*<br>- identifying problems,<br>- giving priorities,<br>- proposing solutions to maintenance or repair. |
| | **5 - Corrective maintenance** | Identification of failure causes from the observed failure (diagnosis of failure). | *Improve **corrective** procedures by:*<br>- explaining failures,<br>- proposing solutions to repair. |

**Figure 10: Different uses of FMEA tool**

### 4.1 Risk analysis

This tool is intended to help industrial partners, to contribute to innovation.

A successful experience with an industrialist was conducted on an innovative cladding system. Two potential failure modes were detected and solved.

Even if the failure modes are uncertain, it is worth pointing out the fact that the product could fail in use. We then analyse its behaviour more accurately using a traditional procedure (artificial testing, natural ageing, etc).

Based on the FMEA study, we know that : "Such materials will be stressed by UV"

By means of traditional methods for durability characterisation, we try to know whether "the product is designed to withstand such stresses ?"

### 4.2 Quality management

FMEA is used as a tool allowing experience and knowledge to be capitalised (as well as the production of information) concerning the frequent defects, errors, negligence occurring during the construction process, expressed in checklists.

For the following three maintenance strategies, other considerations have to be taken into account. For example, the degradation could be accepted because of high maintenance costs.

### 4.3 Preventive maintenance

FMEA produces information concerning the key elements, i.e. elements on which the good working of the product depends (top elements in the failure tree).

This justifies the updating of existing lists of maintenance operations.

Example:

| Product/element | Operation |
|---|---|
| Heavy added protection in a flat roofing system | Limited thickness No vegetation |
| Draining pipes in a wooden window | Preventing water accumulation (rotting of wood pieces) |

**Figure 11: Preventive maintenance**

### 4.4 Conditional maintenance

FMEA is particularly useful for degradation and failure modes with symptoms (warning signs).

With a condition assessment of the building and the products (surveying only the critical elements, guided by FMEA results), we identify symptoms, search for causes and propose relevant cure (maintenance or repair solutions).

We are thus able to prevent costly and/or hazardous failures by identifying warning signs.

Example:

| Symptoms | Cause | Solution |
|---|---|---|
| Water accumulation on a roofing system | Obstruction of draining pipes because of bad maintenance | Cleaning of the draining pipes |
| Degradation (cracking) of seals in a window | Climatic factors | Change of seals |

**Figure 12: Conditional maintenance**

### 4.5 Corrective maintenance (repair)

FMEA is used to explain a degradation and a failure, i.e. to identify the top event and the different events leading to failure, in order to propose relevant repair solutions instead of temporary solutions (searching top event).

Example:

| Failure | Scenario / Top event | Solution |
|---|---|---|
| Air permeability of a window | 1 - Defect in hinges adjustment<br>2 - Wear of seals<br>3 - Air permeability | Hinges adjustment |

**Figure 13: Corrective maintenance**

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## 5 CONCLUSIONS: FMEA – A TOOL FOR QUALITY IN CONSTRUCTION

Based on FMEA, the tool developed in CSTB gives elements to assess the durability of construction products.

On the one hand, we can improve the **reliability and quality of innovative products**. With a risk analysis from the design stage, weak points are identified. We can define relevant preventive actions (risk analysis and quality procedure).

On the other hand, in a "refurbishment/diagnosis" context, the FMEA tool allows the **"in-service follow-up" of existing products** and supplies information for IMR procedures (Inspection/Maintenance/Repair) such as quality control and maintenance planning (conditional and corrective maintenance), failure diagnosis (capability of detecting degradation, prediction of the future degradation or failures, identification and selection of relevant maintenance or repair solutions).

It plays an important part in capitalising on experiments and knowledge within building management.

It is a real advantage for architects, owners, project managers, insurers, users ... to:

- Have a guaranteed quality approach,
- Improve and reveal the quality of products and practices,
- Optimise design costs,
- Communicate reliable information between actors of construction,
- Improve products and practices quality,
- Optimise setting up and maintenance costs,
- Decrease environmental impacts at each stage in the construction process,
- Avoid errors, defects and omissions.

In a context where non-quality or poor-quality costs are being added to the general wish to decrease the global cost of construction, FMECA and management tools should have a determinant role.

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# Permeability And Pore Structure Of Placing Joint Of Cement Mortar Produced In Casting

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**Summary: Permeability of placing joint of cement mortar during casting was measured for the specimens placed at various interval times by using kerosene as permeation liquid instead of water, and a water path in the specimens was observed. The longer placing interval time of the two mortars, the easier liquid leaks through the placing joint. The pore structure was furthermore measured at various positions including placing joint in the specimens by a mercury intrusion method. Pore volume at placing joint is clearly much compared to the other area in the specimen. It is concluded that the existence of porous zone at placing joint results in an increase of permeability.**

**Keywords: Placing joint, Kerosene, Permeability, Mortar, Pore structure, Mercury**

## 1 INTRODUCTION

A concrete member can rarely be placed in one operation so is normally placed in layers with joints in between. Planned construction joints, such as the bottom and top levels of slabs and beams, are sufficiently treated beforehand by, e.g., removing laitance and unsound concrete to integrate the subsequent layer. In normal placing, however, part of concrete may inevitably be placed after a substantial interval without such sufficient treatment where it is supposed to be placed in one continuous operation from the previous layer, due to delayed production and/or transportation. Excessively long intervals result in cold joints, which can be a potential weakness in regard to structural yield strength, while forming a critical waterproofing defects permitting permeation of rainwater through walls and groundwater through underground walls.

According to the Standard Specification for Reinforced Concrete Construction by the Architectural Institute of Japan, the recommended limits of intervals between concreting lifts are 150 min and 120 min at outside air temperatures of lower than 25°C and 25°C or higher, respectively. In practice, the interval may exceed the recommended limit. Even when the interval meets the limit, complete unification cannot always be achieved, as the jointing is strongly affected by the degree of setting of preceding concrete, time from mixing to the end of placing, concrete temperature, and method of consolidation at the joint.

It is clear that placing joints are prone to waterproofing weaknesses, whether or not they are cold joints. This is considered to result from the pore structure of such joints, since the inevitable bleeding water from preceding concrete may alter the microstructure of not only the joint but also the adjacent laminas, and concomitant laitance inhibits unification with the subsequent concrete, leaving large voids in the area. It is therefore essential for grasping the physical properties of placing joints to investigate it in relation to pore structures.

As the first stage of this study, placing joints of mortar were investigated, as the unification of the mortar phase containing plenty of cement paste is critical for actual placing joints, and mortar is easier than concrete to handle. This paper reports on the watertightness of placing joints of mortar discussed from the aspect of pore structures. As the first stage of the study, joints with no tamping through the joint or vibratory consolidation were investigated simulating simple depositing of a mortar layer on the preceding layer.

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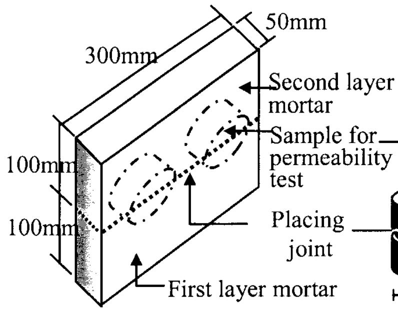
## 2 PLACING JOINT SPECIMENS

As the first step of the study, mortar specimens were prepared. Table 1 gives the mixture proportions, which provide slightly softer consistency than normal mortar to simulate the mortar phase of concrete. Placing joint specimens were fabricated by the following procedure: Mortar was placed in a plywood form with a bottom area of 50 by 300 mm and a depth of 200 mm up to a depth of approximately 100 mm and tamped thoroughly with a tamping rod. The second layer was placed after the specified interval and tamped so that the rod would remain within the second layer(1). Bleeding water from the first layer was not removed to simulate the worst conditions, which can occur in actual walls. As the first step of the study, the joints simulate simple joint conditions without tamping to knit the layers together or vibratory consolidation(2). These specimens were seal-cured in a thermostatic room with a temperature of 20 ± 2°C for 28 days and then dried at a temperature of 20 ± 2°C and a relative humidity of 60 ± 10% for about two months.

Table 1. Mix proportion of mortar

| W/C (%) | C : S | Unit weight (kg/m³) | | | flow (mm) |
|---|---|---|---|---|---|
| | | C | S | W | |
| 55 | 1 :2.6 | 309 | 803 | 170 | 220 |



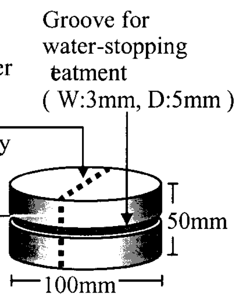




Fig. 1 Mortar specimen Fig.2 Sample for permeability tes

## 3 LIQUID PERMEATION TEST

### 3.1 Specimens for permeation test

Two cylindrical specimens were cut from each of the above-mentioned specimens using a core cutter so that the joint would form axial planes in the centers of the cylinders. Since the cutting involved water, the specimens were dried for another day at a temperature of 20 ± 2°C and a relative humidity of 60 ± 10%.

### 3.2 Test procedure

Each cylindrical specimen was fixed at the bottom of a container for permeation testing. As it was necessary to eliminate leakage through the curved surface of the cylinder, water-stopping treatment was carried out as shown in Fig. 2. A circumferential groove 3 mm wide and 5 mm deep was cut at midheight of the cylinder beforehand. A primer was applied to the curved surface of the cylinder and the side wall of the container. After fixing the cylinder on the bottom of the container, a waterproof sealing compound of rigid polyurethane was poured in the space between the specimen and the container. Being highly flowable and adhesive, this compound was able to perfectly prevent leakage of the test liquid through the vertical surface of the cylinder.

### 3.3 Permeation test using kerosene

Permeation testing using water has conventionally led to unsatisfactory results for the following reasons: It takes long for water to permeate in specimens; It is difficult to obtain accurate data due to the scarceness of permeating water; Measurement may be discontinued by the drying of water, and permeation can completely stop, particularly when the measurement is prolonged, presumably due to resumed hydration of unhydrated cement by the supply of permeating water.

To solve these problems, kerosene was used instead of water in this study. The difference in the dynamic viscosities between water and kerosene, $10 \times 10^{-6}$ m$^2$/s and $1.5 \times 10^{-6}$ m$^2$/s, respectively, naturally leads to different results in absolute values. However, with its readiness to wet mortar (contact angles measured by the authors: 30° for water and 6° for kerosene), kerosene easily permeates through specimens, making permeability testing feasible for specimens having a wide range of permeability, including concrete and mortar having significantly dense microstructures. It is therefore suitable for this study, in which the effects of time intervals between mortar layers are investigated through comparison. The slower drying of kerosene than water also eliminates the problem of drying of the test liquid during measuring. Above all, kerosene has the advantage of causing no hydration in hardened cement during measuring, which is a matter of haunting concern when water is used as the test liquid, ensuring stable measurement throughout a long test period. Kerosene was filled in each container having a fixed specimen as shown in Fig.3, to which a gas pressure of 2 kgf/cm2 (0.196 MPa) was applied from a nitrogen gas cylinder. All measurements were made in a thermostatic room with a temperature of 20 ± 2°C and a relative humidity of 60 ± 10%.

## 4 MEASUREMENT RESULTS

When the pressure was applied, kerosene permeated the specimen along its joint (Photo.1) and immediately spread over the bottom of the specimen. Droplets of kerosene swelled mostly along the joint and finally dropped. Typical results are shown in

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Fig. 4. Though the rate of permeation was unstable in the beginning, it converged over time to a certain value. Since this steady permeation of kerosene was considered a steady state, the permeability coefficient was determined based on the rate of permeation at this stage. As the permeation concentrated along the joint, it was difficult to evaluate it per unit area. A permeability coefficient was therefore determined per unit length of a joint as follows:

$$K_{pj} = Q\,\rho d / pl \qquad (1)$$

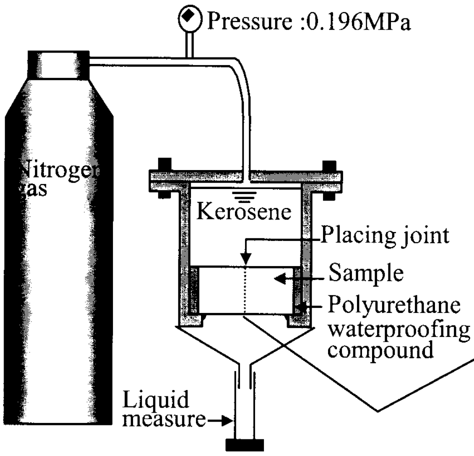


Fig.3 Apparatus for permeability test

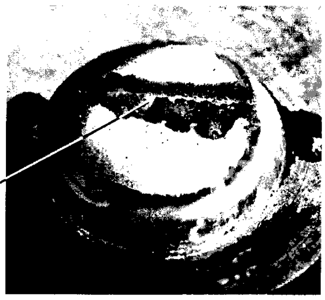

Photo.1 Kerosene permeation through joint

where $K_{pj}$ : permeability coefficient perunit length, $cm^2/s$

Q : quantity, $cm^3$

ρ : density, $g/cm^3$ (kerosene: $0.8 g/cm^3$)

p : pressure, Pa

d : thickness of specimen, cm

l : length of joint, cm

Figure 5 shows the relationship between the permeability coefficient and the time interval between mortar layers. The permeability coefficient tends to increase as the interval increases and increase further after 3hrs. It then significantly increases at 12hrs from the beginning.

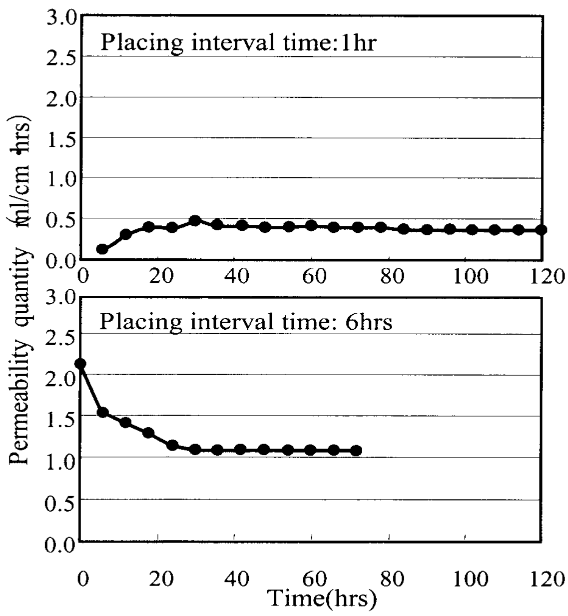


Fig.4 Changes of permeated quantity of kerosene

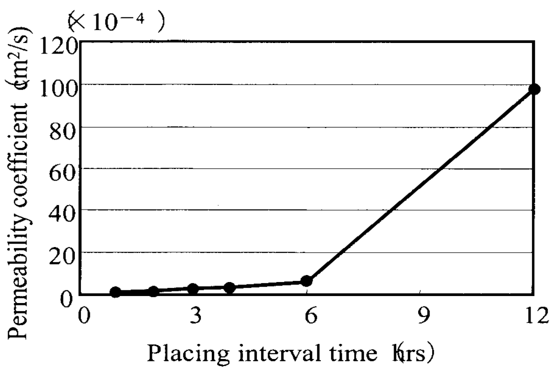


Fig.5 Relation between permeability coefficient and placing interval time

## 5 OBSERVATION OF PERMEATION PATHS

The permeation of specimens with joints was observed to find the permeation paths.

### 5.1 Observation procedure

Immediately after the permeability testing was completed, kerosene was discharged from the container, and an aqueous solution of a fluorescent dye (uranine) was added. A pressure of 5 $kgf/cm^2$ (0.49 MPa) was then applied. After confirming the dropping of uranine along the joint at the bottom using a black lamp as shown in Photo.2, each specimen was axially cleft into two along a plane perpendicular to the joint plane for observation of the uranine permeation paths within the specimen under black lamp.

Paths of permeation are normally observed by pressing in an aqueous solution of a fluorescent dye from the beginning. However, it causes the dye solution to enter pore systems other than the actual permeation path by being absorbed by mortar. The fluorescent solution spreads over the top surface and near the permeation paths, hampering recognition of the actual permeation path. The method proposed here, in which kerosene is allowed to permeate first, followed by an aqueous solution of a fluorescent dye, has an advantage that the permeation path is clearly recognizable, as small pore systems have already been filled with kerosene, which is hydrophobic, preventing the fluorescent solution from entering. As a result, the fluorescent solution is limited to the actual path of liquid permeation, permitting clear recognition of the path.

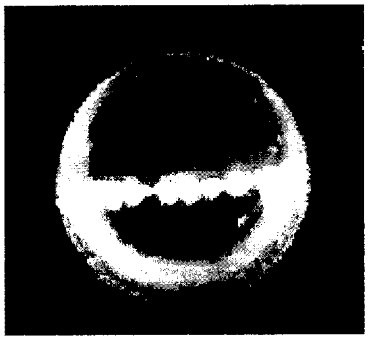

Photo.2 Shining of droplets of fluorescent dye solution along joint under black lamp

### 5.2 Results of permeation path observation

Photo.3 shows the permeation paths of the fluorescent dye solution. The nearly linear luminous trails through the specimens clearly indicate the permeation of the fluorescent solution along the joints.

### 5.3 Relationship between time interval between layers and width of permeating path

While the time interval between mortar layers is short, the width of the permeation path is wide. As the interval increases, the permeation path becomes narrower and sharper. In other words, when the interval is short, the liquid permeates through a slightly wider zone, but as the interval becomes longer, the liquid tends to pass through a narrower but predominant path.

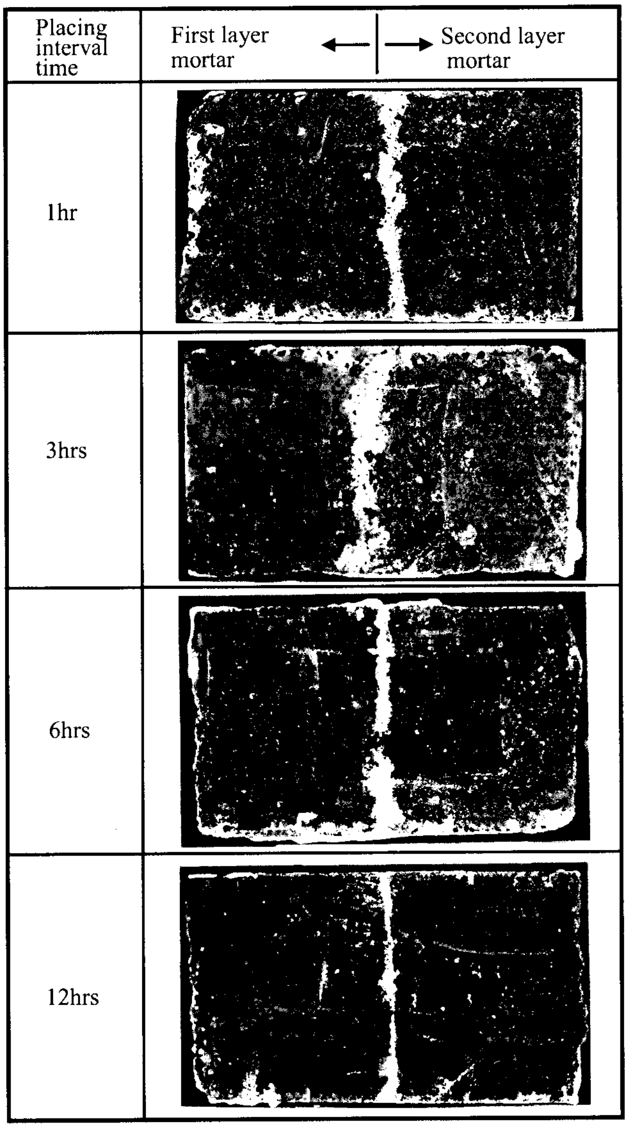


Photo. 3 Permeation paths of the fluorescence dye solution

## 6 PORE STRUCTURES IN AND NEAR PLACING JOINTS

### 6.1 Specimens

When taking samples for pore structure specimens of placing joints, it is essential to know the accurate location of the placing joint surface. A joint with a long time interval is readily recognizable, but recognition of a joint with a short interval is quite difficult. For this reason, transparent acrylic molds were used to allow recognition of the joint during placing, and specimens with clear indication of the joint position were fabricated.

The acrylic molds measured 100 mm in diameter and 200 mm in length, in which mortar was placed with a joint by the same procedure as the specimens for permeation testing. These were seal-cured at a temperature of 20 ± 2°C for 28 days, but the drying process was omitted. The conditions of the specimens are therefore not exactly the same as the permeation specimens, but the omission of the drying process in the fabrication stage may not significantly affect the results, since D-drying is included in the procedure of preparing specimens for pore measurement.

### 6.2 Pore structure measurement

After seal curing, the central portion of each cylinder was roughly cut out using a concrete cutter, from which thin leaves with an area of 15 mm by 10 mm and a thickness of 1.5 mm (3 mm for the leaf including the joint) were cut using a ceramic cutter.

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Nine leaves were taken from each specimen: Five from the center (the leaf including the joint and two adjacent leaves each above and below that, totaling 4.5 mm each above and below the joint) and one each from the top, bottom, middle of the first layer (about 50 mm from the bottom), and middle of the second layer. The pore distribution in the range of 10 µm to 160 µm was measured by mercury intrusion using a pore sizer 9310

### 6.3 Measurement results and discussion

### (1) Pore structure

Typical measurements are shown in Fig. 6. The pore structure of the leaf including the joint is evidently coarser than other portions. The pore structure in the first layer is coarser than that in the second layer. The differences are particularly evident in the pore diameter range of 0.05 to 1 µm, the so-called capillary void range. Figure 7 shows the distribution of volumes of pores greater than a diameter of 0.05 µm for each time interval between mortar layers[3]. This figure reveals that the pore volume of mortar in the first layer is greater than in the second layer, indicating coarser pore structure in the first layer than in the second layer. This tendency is more evident near the joint, presumably due to the rising of bleeding water.

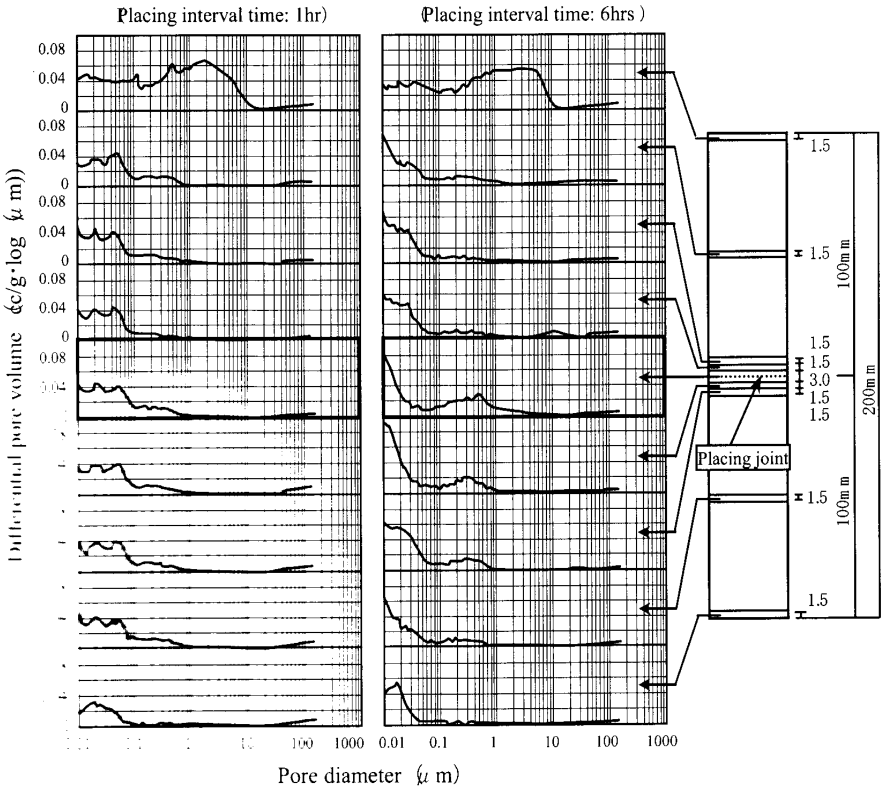
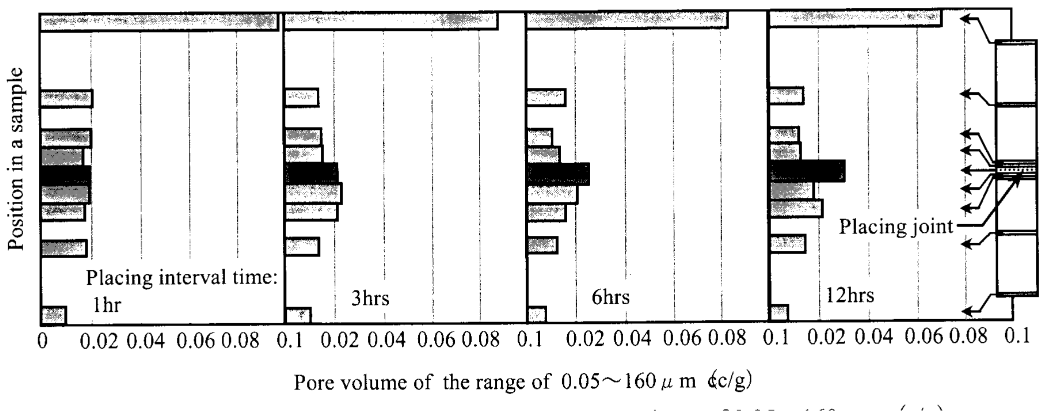



Fig.6 Pore distribution of the samples taken from various positions of the specimens



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## (2) Effect of time interval between mortar layers

Figure 8 shows the relationship between the volume of pores greater than 0.05 µm and the time interval between mortar layers. As the interval increases, the pore volume in this diameter range evidently increases, i.e., the pore structure becomes coarser.

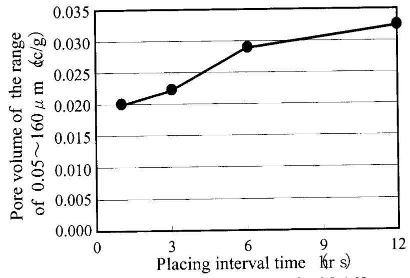


Fig.8 Change of the pore volume of 0.05-160 µm in the leaves including placing joint with placing interval time

## (3) Pore structures of joints with particularly long time intervals

Since joint samples having a certain thickness were used to measure the pore volume, the measurements do not represent only the interface but the total value including its vicinities. However, specimens with a particularly long time interval between the layers (6 and 12hrs) can be easily separated into the first and second layers of mortar, providing information of the interface alone. The idea is illustrated in Fig. 9.This is begun by measuring the pore size distribution of the 3-mm sample including the joint.

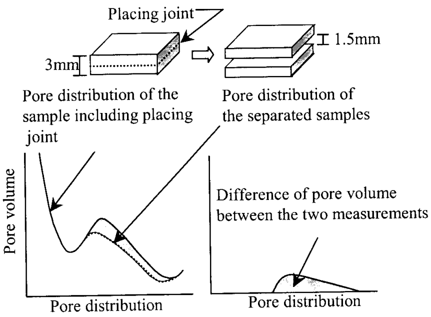


Fig. 9 Idea for estimating of pore distribution of placing joint interface

The sample is then separated at the interface into two layers, and the pore size distribution of each layer is measured. The difference between the measurements is examined. This difference is considered to represent the pore structures of the interfaces. Figure 10 shows the pore structure of the interfaces estimated according to this idea. This figure clearly exhibits that pores between the orders of 1 µm and 10 µm are present along the joints. It is naturally expected that the case of 12-hours interval should involve larger pores, but such sizes are beyond the range of the mercury intrusion measurment. No information as to the large size range is therefore obtained unfortunately.

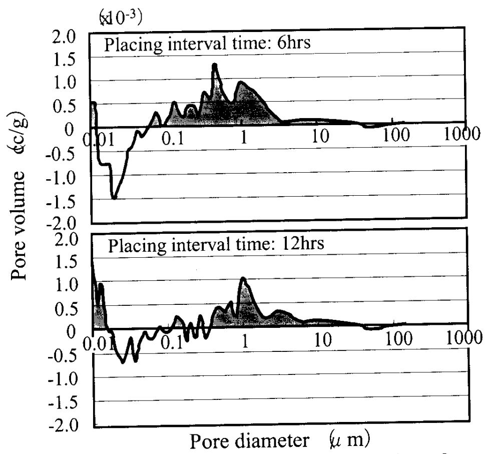


Fig. 10 Estimated of pore distribution of placing joint interfaces

## 7 RELATIONSHIP BETWEEN PERMEABILITY OF PLACING JIONTS AND PORE STRUCTURE

Figure 11 shows the relationship between the permeability coefficient and the pore volume in the range of 0.05 to 160 µm. As the time interval between mortar layers increases, the pore volume increases and the permeability coefficient monotonically increases. In other words, the increase in the permeability of the placing joint is considered to result from the increase in the volume of pores with a diameter in the capillary range or greater. As stated in the discussion of the pore structure on the joint surface in the previous section, presence of pores of a particularly large range is expected in the case of a 12-hour interval, causing significantly high permeability of the joint zone.

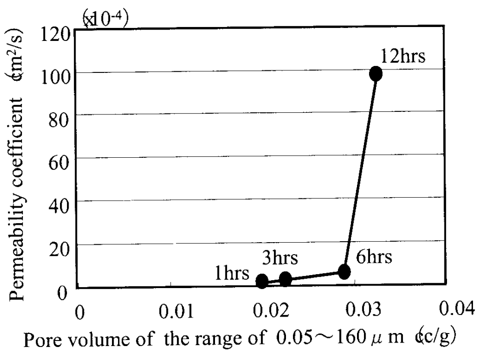


Fig. 11 Relation between permeability coefficient and pore volume

## 8 CONCLUSIONS

This study led to the following conclusions:

(1) The permeability of joint specimens made with a short time interval between layers can be measured by using kerosene instead of water.

(2) As the time intervals increased, the permeability increased. The permeability of 12-hour interval specimens was particularly large.

(3) The path of the test solution is limited to the joint plane and its close vicinities.

(4) The microstructure of a placing joint and its vicinities is coarser than in other portions. This tendency becomes more evident as the time interval at the joint increases.

It should be noted that, in this study, the joints between mortar layers were intentionally retained by avoiding unification by tamping to knit both layers together. In actual construction, concrete is normally unified by tamping or vibratory consolidation. The effect of such unifying action is the subject for our following study.

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# Long-term Performance Evaluation Of External Renderings On Autoclaved Aerated Concrete Wall

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**Summary: The use of external renderings and surface treatments mainly aims to improve the weather resistance of external walls. The influence of agents present within the microenvironment, particularly moisture, causes degradation of external walls and thus reduces their performance over time. In order to study the long-term effects on rendered autoclaved aerated concrete (AAC) walls, an experimental building has been built. The microenvironment parameters have been continuously measured and laboratory tests and analysis have been carried out on sample cores taken each year from test panels mounted on the experimental building. In this paper, the long-term performance of external renderings is evaluated based on the comparison of test results of the capillary water absorption of sample cores and the microenvironment measurements. The first results obtained after a two-year exposure period indicate that inorganic renderings have improved their initial moisture performance while the performance of organic renderings remained about the same.**

**Keywords: AAC, external rendering, long-term performance, moisture.**

## 1 INTRODUCTION

Rendered AAC is one of the alternative wall construction types to external walls with insulation systems. Because of the low thermal conductivity of AAC, additional insulation may not be required depending on the climate where the construction is built and on the use of the building. In addition to energy savings during use, it could also save material and reduce work during the construction phase. Moreover, complex moisture problems of multi-layered walls can be avoided by minimising the number of layers. However, as for other wall types, protection of AAC against outdoor conditions, particularly rain, plays an important role for the service life. The durability and service life of rendered AAC walls depends on factors such as characteristics of the building materials forming the wall components, design and the application (workmanship) level, environmental factors and maintenance conditions (ISO 15686-1). Most factors can be controlled, however the exposure environment remains as an important factor, which should be primarily considered in performance assessment. Different coatings and external rendering systems are applied to improve resistance to environmental factors. In fact, performance of these surface coatings and renderings determines the long-term performance of the whole wall component.

In order to study the actual in-service performance of rendered AAC walls, an international research programme, EUREKA-project DurAAC E 2116 (Kus & Nygren 2000), has been initiated at the Centre for Built Environment in Gävle, Sweden. The long-term effects of different rendering systems on AAC are investigated by long-term moisture monitoring at the test cabin and by periodical laboratory tests as well as analysis of sample cores taken from the test cabin every year over a five-year period. The measuring (exposure) programme started in June 1999. In this paper, different rendering systems on AAC are evaluated, based on the comparison of test results of the capillary water absorption of sample cores and the data obtained from the continuous electrical measurements of moisture at the test cabin.

## 2 EXPERIMENTAL

### 2.1 Materials

Un-reinforced AAC panels with dry density 423 kg/m$^3$ and measuring 600 mm × 1200 mm × 150 mm were used as substrates for the rendering systems tested. The rendering work of the test panels was performed indoors by a skilled building contractor under controlled conditions. In this way, the experimental set-up would be repeatable and the performance of the test panels would be comparable. The applications of renderings and water repellants followed the manufacturer's instructions. After a curing period of approximately three months, rendered test panels (Fig. 1 and 2) were installed to the long sides of the test cabin facing north-east and south-west, respectively. There are 12 test panels on each of the two facades including uncoated and un-rendered AAC as control sample panels, System 1. The joints between the test panels themselves and the wall were insulated and sealed. The systems tested involve inorganic and organic renderings as well as their modifications with water repellants. The compositions of the six rendering systems, which are studied in this paper, are summarised in Table 1 below.

**Figure 1. Test panels**

**Figure 2. Test panels from the inside**

**Table 1. Rendering Systems**

| *System No* | *AAC Surface Impregnation* | *Primer* | *Undercoat* | *Final Coat* | *Thickness* (mm) |
|---|---|---|---|---|---|
| 5 | - | Lime/white cement/dolomite 10/90/350 | - | Lime/white cement/white dolomite 50/50/450 | 5-7 |
| 6 | - | System No 5 *with silicon additive* | - | System No 5 *with silicon additive* | 5-7 |
| 7 | - | Lime/white cement/dolomite 10/90/350 | Lime/cement/ dolomite-sand 50/50/650 | Hydraulic lime/ lime/dolomite | 10-12 |
| 9 | Silane-siloxane emulsion | Acrylic/dolomite-calcite *with silicon additive* | - | Pure acrylic copolymer/dolomite-calcite | <1 |
| 11 | Silane-siloxane emulsion | - | - | Silicon resin | <1 |
| 12 | - | - | Cement/polymer/lime stone-plastic fibres | Cement/polymer/lime stone-plastic fibres | 3-4 |

## 2.2 Measurements

Measurements taken can be grouped as continuous field measurements, periodical field measurements, and laboratory tests and analysis of material properties and over time. The programme will run for at least five years.

### 2.2.1 Continuous Microenvironment Measurements

Continuous field measurements are carried out at five-minute intervals and hourly averages are registered. The parameters monitored are:

- Surface and bulk moisture of the materials
- Surface and bulk temperature of the materials
- Indoor and outdoor air temperature
- Indoor and outdoor relative humidity
- Driving rain
- Ultraviolet radiation.

Moisture content of the test panels exposed at the test cabin have been monitored since the start of the exposure programme. Wetcorr sensors and resistance-type nail electrodes are used to measure both the surface moisture and the bulk moisture of the material. Resistance is measured by epoxy coated nail electrode pairs placed at seven different depths (in the rendering and in the AAC) in addition to temperature measurements with copper-constantan-type thermocouples at the same depths. The moisture sensors and thermocouples are connected to the terminals of five multiplexers controlled by two dataloggers. The resistance measured between the two electrodes together with the temperature, both in the AAC material and in the thick rendering, was converted to moisture content by means of a calibration procedure (Kus & Norberg 2001). Since the moisture performance of AAC is directly related to the effectiveness of the rendering system, the assessment is based on the electrical measurements of moisture made in the AAC.

#### 2.2.2 Capillary Water Absorption Test

Once a year, three sample cores were drilled (with water) from each panel of the test cabin. The diameter of each core was 90-95 mm and the thickness approximately 150 mm (AAC and rendering system). The samples were then conditioned at 20 °C and 65 % relative humidity. After drying in the oven at 65 °C, surfaces of the samples, except for the exposed surface, were sealed with wax. The samples were immersed, exposed faces down into 1-3 mm of distilled water and were then weighed at various time intervals. For the initial test three samples were tested, whilst only one was tested after the first year of exposure and two after the second year.

## 3 RESULTS AND DISCUSSION

### 3.1 Microenvironment Measurements

The weather conditions at the north-east facade for the selected periods are summarised in Table 2. These conditions should be taken into consideration in order to make a better assessment of the rendering systems. For example, the total amount of driving rain is much higher during the periods in the first two years compared to the amount during the period in the last year. However, the average relative humidity is the highest during the period in year 2000 while the highest average temperature is found during the period in year 2001.

**Table 2. Microenvironment measurements at the north-east facade**

| | *unit* | *5-12 Aug 1999* | *29 Jun-6 Jul 2000* | *20-27 Aug 2001* |
|---|---|---|---|---|
| Air Temperature (average) | °C | 14,7 | 14,7 | 17,8 |
| RH (average) | % | 74 | 83 | 77 |
| Driving rain (total amount) | $kg/m^2/h$ | 9,1 | 9,9 | 1,6 |

In Figures 3-5, air temperatures, relative humidity and the driving rain monitored on the north-east facade are shown for selected periods. The indoor air temperature is kept at minimum of 15 °C through the year. Because the south-west facade is subjected to sunlight over longer periods, the test panels demonstrate relatively low moisture contents and therefore are not included in this paper. In order to get the most significant effects, the most rainy and humid weather conditions were selected. The ageing process for different rendering systems can be analysed from initial data and data obtained at yearly intervals.

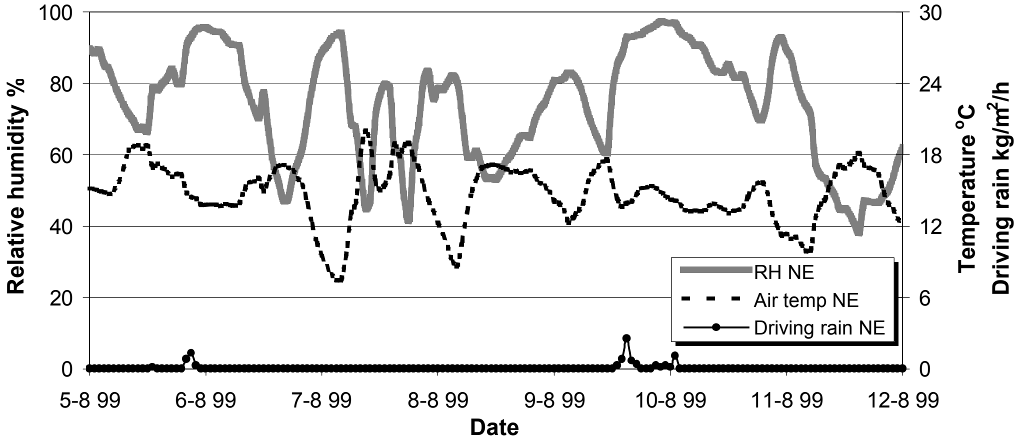


**Figure 3. Microenvironment measurements: NE facade: 1999**

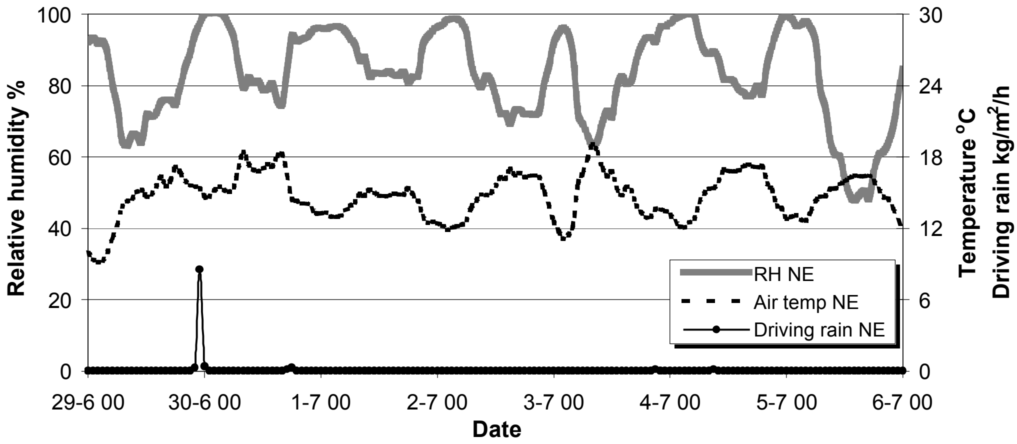


**Figure 4. Microenvironment measurements: NE facade: 2000**

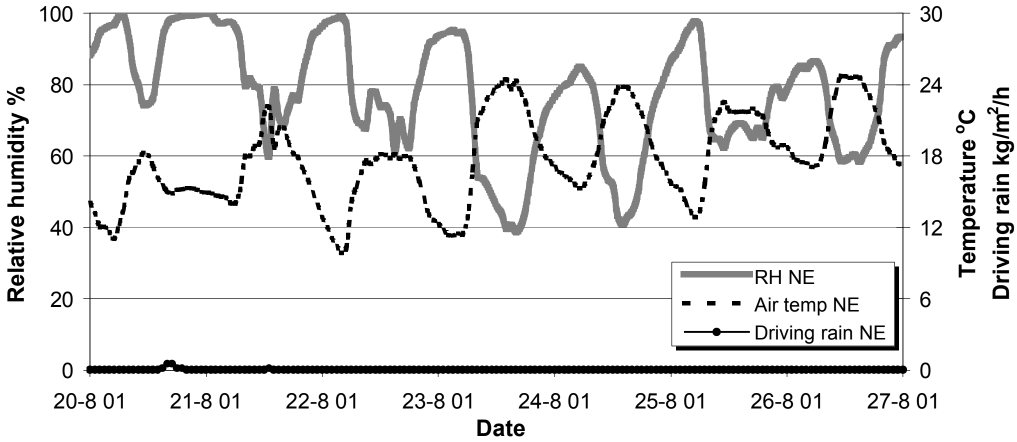


**Figure 5. Microenvironment measurements: NE facade: 2001**

## 3.2 Continuous Moisture Measurements

The average moisture content from one-week data measured at two different depths in the AAC is displayed in Figures 6 and 7. The moisture performance of lime-cement rendering systems, *Systems 5* and *7*, differs most from the other systems. The moisture content in these systems with inorganic renderings is higher than the control panel (plain AAC), *System 1* (Fig. 6). The only impairment over time, although rather small, appears for *System 6*. The performance of both *Systems 9* and *11* seems unchanged during the whole exposure period. The high moisture content of *Systems 5* and *7* at 25 mm depth in the AAC (Fig. 7) probably depends on the intensity of the driving rain and the high relative humidity during this period.

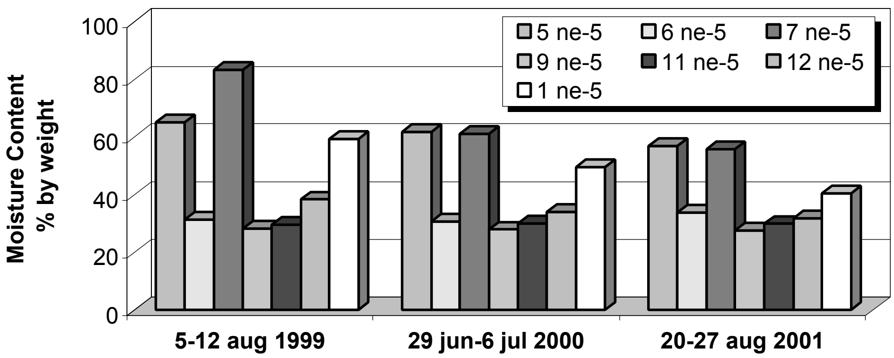


**Figure 6. Average moisture content at 5 mm depth**

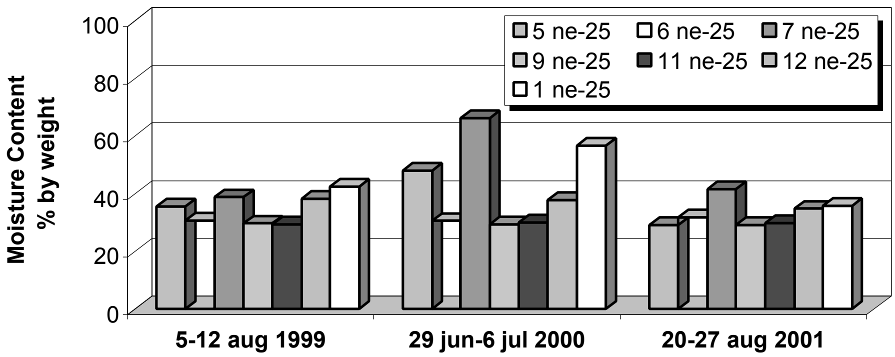


**Figure 7. Average moisture content at 25 mm depth**

3.2.1 At 5 mm depth from the AAC surface

The rendering systems modified with water repellants, *Systems 6*, *9* and *11*, and the fibre reinforced cement-polymer rendering, *System 12*, had relatively low moisture contents compared to the lime-cement rendering systems, *Systems 5* and *7* (Fig. 8-10). Initial measurements indicate that both thin and thick lime-cement rendering systems, *Systems 5* and *7*, absorbed about the same amounts, however, the absorption rate of *System 7* was lower, probably because of the thickness of the rendering which is thicker than that of *System 5* (Fig. 8). Also, it took longer for *System 7* to dry out and it absorbed moisture later and the moisture content remained higher than that of *System 5* when the second wave of driving rain occured (Fig. 8). After the first and second year of exposure, thick lime-cement rendering, *System 7*, absorbed less than the thin lime-cement rendering, *System 5*, however, demonstrated almost the same rate of drying (Fig. 9-10). When the duration of driving rain was very long, the lime-cement rendering containing silicon additive, *System 6*, and the fibre reinforced cement-polymer rendering, *System 12*, began to absorb small amounts of moisture. This is clearly seen, particularly after the second driving rain period (Fig. 8). This implies that the effectiveness of water repellent additives decreases when they are exposed to longer periods of driving rain.

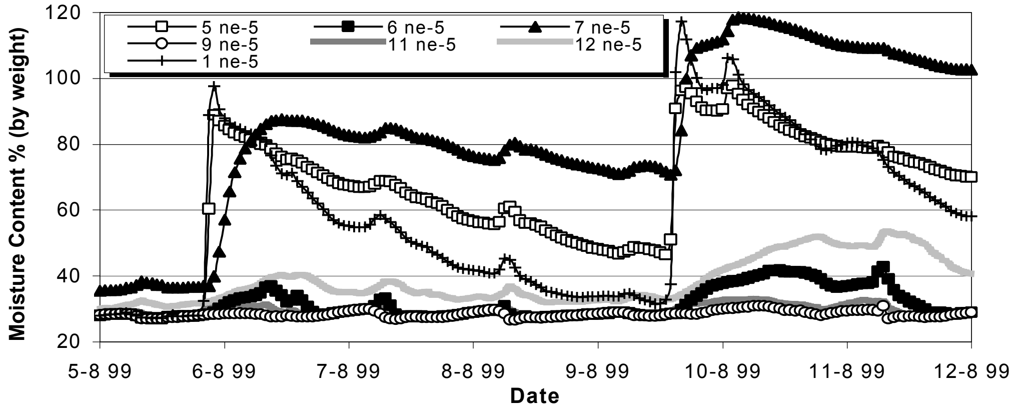


**Figure 8. NE facade, 5 mm depth from the AAC surface: 1999**

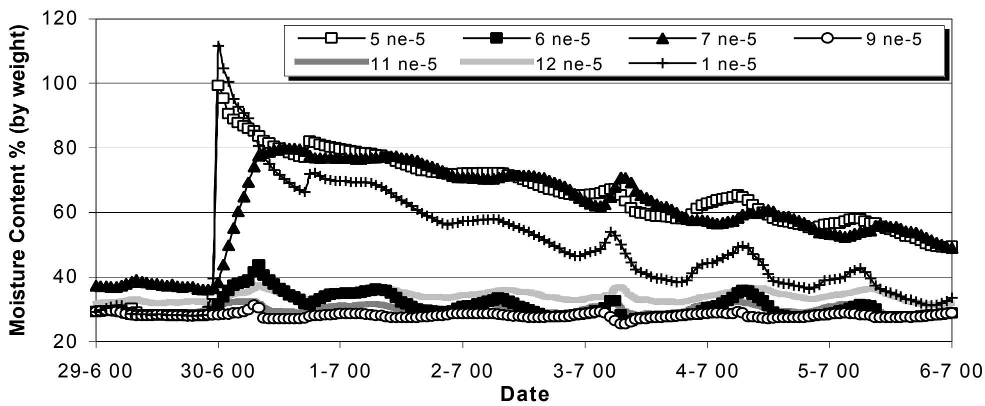


**Figure 9. NE facade, 5 mm depth from the AAC surface: 2000**

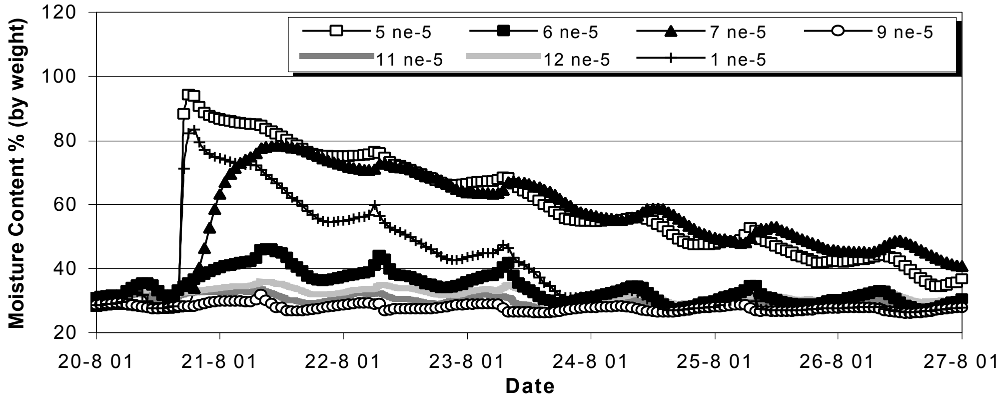


**Figure 10. NE facade, 5 mm depth from the AAC surface: 2001**

3.2.2 At 25 mm depth from the AAC surface

After approximately one year of exposure, the test panels with inorganic renderings, *Systems 7* and *5*, and the control panel (plain AAC), *System 1*, had the highest moisture contents at 25 mm depth (Fig. 12). This might be due to the intensity of the driving rain during this period which probably propagated the capillary suction inwards.

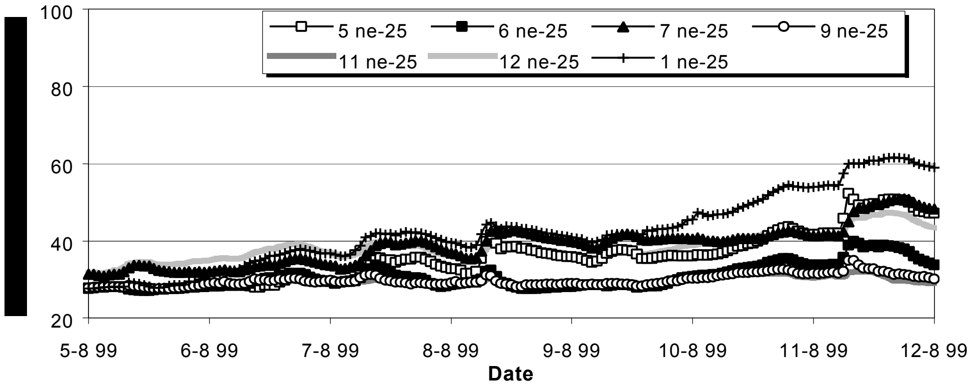


**Figure 11. NE facade, 25 mm depth from the AAC surface: 1999**

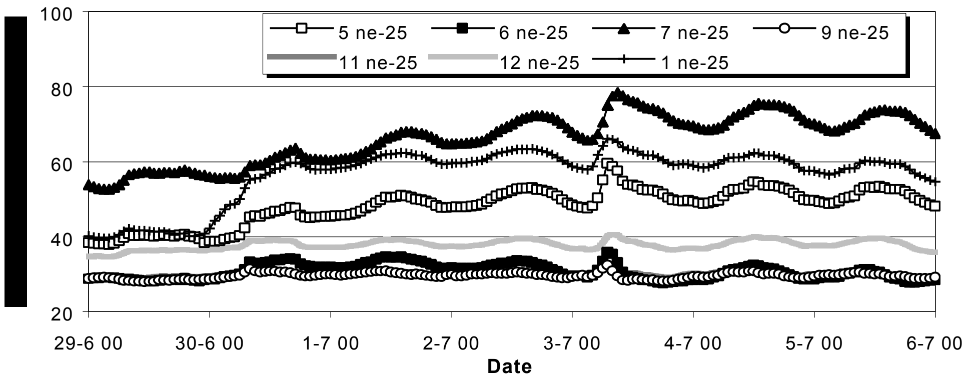


**Figure 12. NE facade, 25 mm depth from the AAC surface: 2000**

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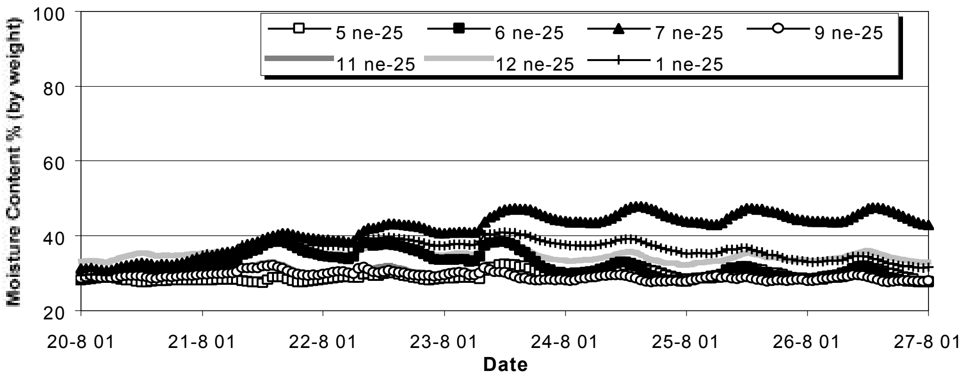


**Figure 13. NE facade, 25 mm depth from the AAC surface: 2001**

## 3.3 Capillary Water Absorption

Figures 14-16 demonstrate the capillary water absorption of the sample specimens, initially and over time. In general, the results indicate similar long-term declining tendency in suction properties of the inorganic systems, *Systems 7* and *5*. The fiber reinforced rendering, *System 12*, shows similar moisture behaviour over time, which is also decreasing. *Systems 9* and *11* keep almost the same low absorption levels after the second year of exposure (Fig. 16). The curve for *System 6* confirms its susceptibility to moisture absorption when it is in contact with water for a long duration, however the absorption rate is low compared to the inorganic systems.

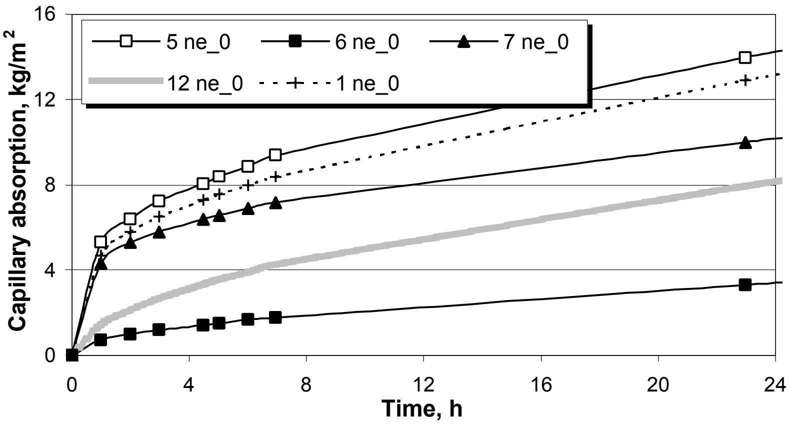


**Figure 14. Initial capillary absorption: 1999**

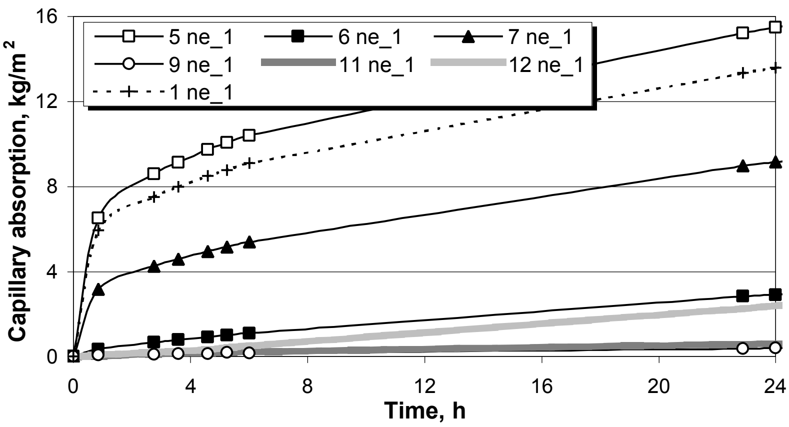


**Figure 15. Capillary absorption: 2000**

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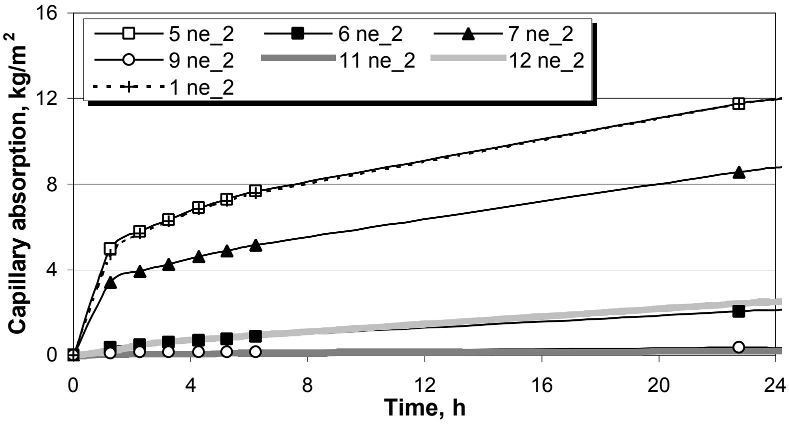


**Figure 16. Capillary absorption: 2001**

## 4 CONCLUSIONS

The long-term performance of different external rendering systems on AAC is being investigated within an international research project. The research programme consists of continuous microenvironment measurements and laboratory tests and analysis of initial material properties and material properties over time. Twelve systems including inorganic renderings and organic renderings as well as their modifications with silicon-based water repellants have been investigated. Efficiency of the rendering systems is basically assessed by moisture analysis of naturally exposed test panels mounted on an experimental building. The degradation processes and ageing characteristics are also studied by microstructure analysis of sample cores taken from the experimental building each year over a five-year period.

Among the six systems studied in this paper, *Systems 9* and *11*, the ones with impregnated AAC surface, perform best so far. Due to the hydrophobic effect of silicones, almost no change in moisture content has been observed during the initial two-year exposure period. The moisture content of *System 6*, the thin lime-cement rendering containing silicon additive, only increases after a long duration of driving rain. During long periods of driving rain the moisture content of *System 6* reached up to 80 % by weight. On the other hand, its drying rate was high. Initially, *System 12*, fibre reinforced rendering, was fairly susceptible to driving rain while subsequently improving its performance. *Systems 5* and *7*, the two inorganic systems, demonstrated the highest moisture absorption rates. However, after a short exposure period, the moisture contents of both inorganic systems decreased to some extent. The drying rate of *System 7* was lower compared to that of *System 5.* Faster degradation can be expected for *System 7* due to high moisture content during long periods of time.

First results obtained after a two-year exposure period indicate that inorganic renderings have improved their initial performance while the performance of organic renderings remained about the same. Another preliminary conclusion is the susceptibility of lime-cement renderings containing silicon additive to driving rain of long duration. Overall, the first results from the moisture monitoring give some insight into the actual in-service performance of different rendering systems on AAC. However, data from the two-year exposure period is not sufficient for reliable evaluation of long-term performance. The research programme is to be carried out for at least five years. Hence, it is expected that clearer findings will be made.

## 5 ACKNOWLEDGEMENTS

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# Performance Characteristics' Degradation And Paints' Protective Degree For Wall Components Subject To Durability Tests

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**Summary: The paper reports on the conclusions obtained at the end of the first experimental part of the research on building components durability, which is now being developed in cooperation between DISET Politecnico di Milano and Laboratorio Tecnico Sperimentale of the University of Applied Sciences of Southern Switzerland.**

**The reported analysis are related with the measurement of the time trend of performance characteristics measured on external wall components, with and without painting protection, after laboratory weathering cycles. The comparison between results obtained on samples protected by two different paintings, one with acrylic resin and the other with vinyl resins, and different Pigment Volume Concentration of 45% and 60%, with respect to the reference unprotected samples, allows to evaluate the protective degree for the tested finishing layers. The analysis of time trends of measured performance characteristics, eventually compared with each other allows to achieve the first evaluations of building component durability.**

**The measured performance characteristics concerns with water transmission properties (capillary absorption and vapor diffusion) For the service life evaluation these analysis are completed with natural weathering tests' results, in Milano (I) and Lugano (CH) to establish the correlation between accelerated aging time and natural aging time.**

**Besides the correlation between the materials' performance characteristics and the component's actual performance decay, in realistic conditions will be estimated during testing on the complete external wall component subject to differential stressing conditions, simulating indoor and outdoor environments.**

**At the moment we have obtained useful information about the comprehension of degradation phenomena and about time behavior of materials constituting the building component subject to reference laboratory accelerated aging conditions.**

**It's shown in the report also some result relating with the time re-scaling, demonstrating that the outdoor samples show the same damages we obtained in laboratory: we are able now to evaluate the time re-scaling to estimate the Reference Service Life of the component.**

**Keywords: components, materials, performance, laboratory, paints**

## 1 INTRODUCTION

The Department of Engineering of Building Systems and Urban Planning (DISET) in Politecnico di Milano has been developing through the last years research on technological quality of building components, mainly regarding the durability and reliability evaluation (Maggi *et al.* 1987, Maggi *et al.* 1990, Daniotti *et al.* 1994).

After the first methodological studies the research brought to an experimental program for the laboratory evaluation of building components' durability, developed in cooperation between DISET Politecnico di Milano and Laboratorio Tecnico Sperimentale of the University of Applied Sciences of Southern Switzerland (Daniotti *et al.* 1998).

The first experimental phase of the research is aimed to apply the general methodology for the evaluation of durability to an external wall's component (Rigamonti 2002).

The reported conclusions and results regard the laboratory aging of the external layers' packet of the wall. The analysis of time trends of measured performance characteristics, eventually compared with each other allows to achieve the first evaluation of building component durability.

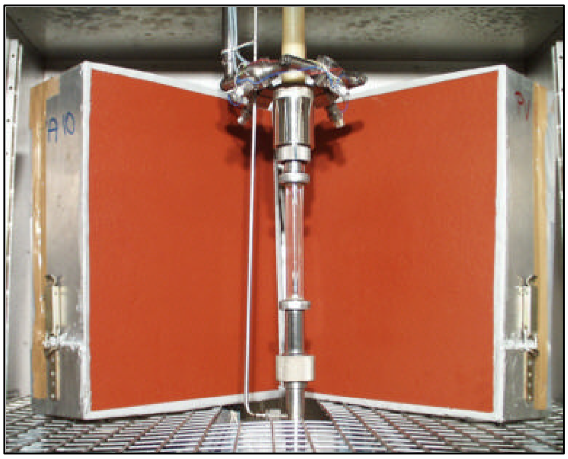

**Figure 1. Test samples during aging**

The aging cycle has been divided into four phases representing different extreme weathering conditions on the base of relevant agents (Table 1).

**Table. 1. Aging cycle's phases**

| | Temperature (°C) | Relative Humidity (%) | Time (minutes) |
|---|---|---|---|
| RAIN | 20 | 95 | 60 |
| FREEZE | -20 | - | 90 |
| HOT – HUMID | 55 | 95 | 60 |
| HOT – DRY with UV artificial sunlight | 30 | 40 | 80 |

The measured performance characteristics concern with water transmission properties (capillary absorption and vapor diffusion), thermal transmission properties (thermal resistance), and mechanical properties (compression, tensile and flexural resistances); the chemical – physical degradation of materials constituting the wall component solution has been analyzed through surface macro photos, microscope observations and porosity measurements. (Teruzzi and Jornet 2002).

## 2 MASS INCREASE

A first important result regards the mass increase of samples due to the water cumulating during aging cycles, as a function of different protective degree of the tested paints; the measured mass increase over time for the different protected samples are compared in Fig.2.

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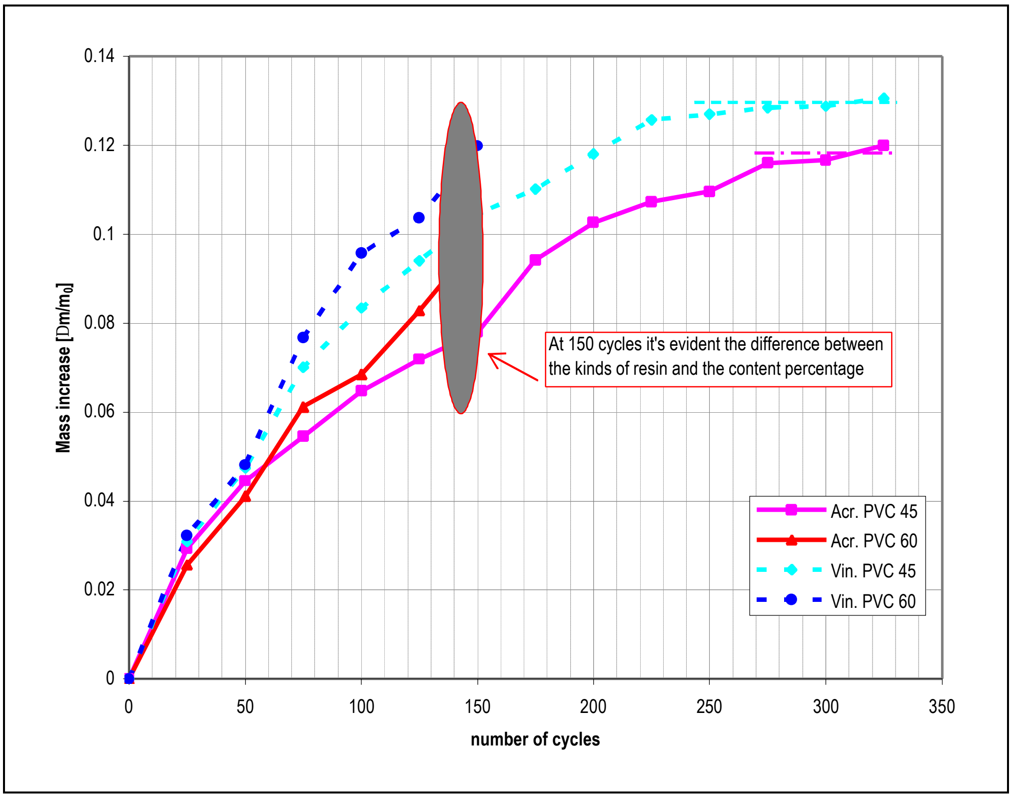


**Figure 2. Mass increase as a function of aging cycles number.**

Looking at the graph we can draw the following conclusions:

- Already at 150 aging cycles we find relevant differences among different kind of paints, that allow us to evaluate their protective degree to water penetration.
- From the lower to higher we have firstly the acrylic paints, at a higher resins' content and at low resins' content, and then the vinyl resin's paints at a higher resins' content and at low resins' content.
- Considering the percentage difference between the mass increase at 150 cycles for each paint and the best acrylic resin's paint PVC 45 (0.078 $\Delta m/m_0$) we have:
    - +27 % acrylic resin's paint PVC 60
    - +34 % vinyl resin's paint PVC 45
    - +54 % vinyl resin's paint PVC 60
- The time trend of mass increase is asymptotic: all samples reach an equilibrium situation for water transfer in the raining and drying phases: the different behavior of paintings in these two phases will be later discussed in the analysis of capillary absorption, prevailing in the raining phase as liquid status and in the analysis of water vapor diffusion coefficient, prevailing during drying phase.
- The differences between the different samples are evident starting from 75 aging cycles, which can then be considered a minimum time limit for this test.
- The application of an aging test until 150 cycles allows to obtain a good estimation of protective degree for these paints.

## 3 WATER ABSORPTION

The second important result is related to the elaboration of water capillary absorption data for the various paints. To evaluate the painting's protective degree to water we have taken away a 3 mm surface containing the protective layer and then we have repeated the water capillary absorption test. In the histogram in Fig 3 we have reported the percent difference between the capillary absorption of the protected sample and the samples after the uptake of the protective layer, for the three measurement times: 10 minutes, 1 hour and 24 hours.

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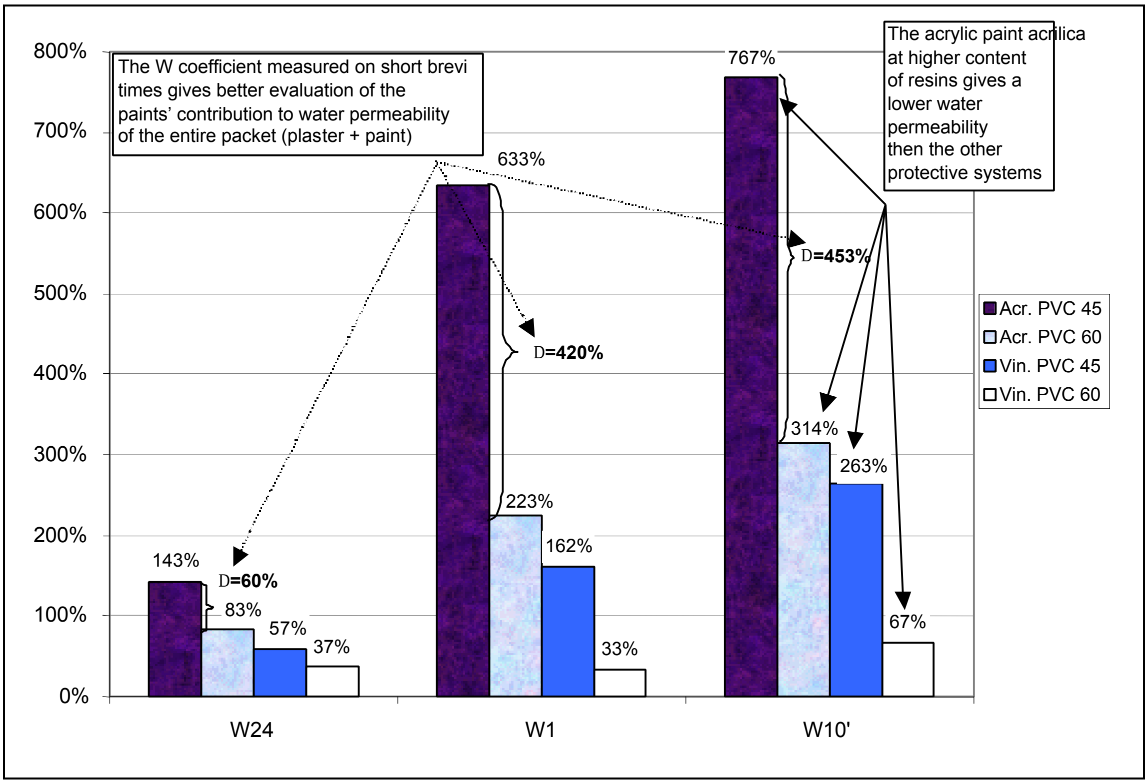


**Figure 3 Graph of percent difference of capillary absorption between painted sample and plaster without painting protection.**

We can see that at 10 minutes and 1 hour we have the maximum differences in the paints' behavior in terms of capillary absorption, while at 24 hours the external layers of painted plaster gives a global contribution to the water absorption: the data at 10 minutes and 1 hour is then the most interesting to give an evaluation for the protective degree to water.

Comparing the different paintings we can perceive that the best protective degree is given by the acrylic paints with respect to the vinyl resin's ones and that the higher content of resin contributes to obtain the best performance. The acrylic paint at higher resin's content gives the best results as water absorption differences with respect to the unprotected plaster.

## 4 WATER VAPOR PERMEABILITY

Water vapor permeability and the corresponding resistance to water vapor diffusion were determined on three cores for each wall specimen and the results are shown in Fig. 4.

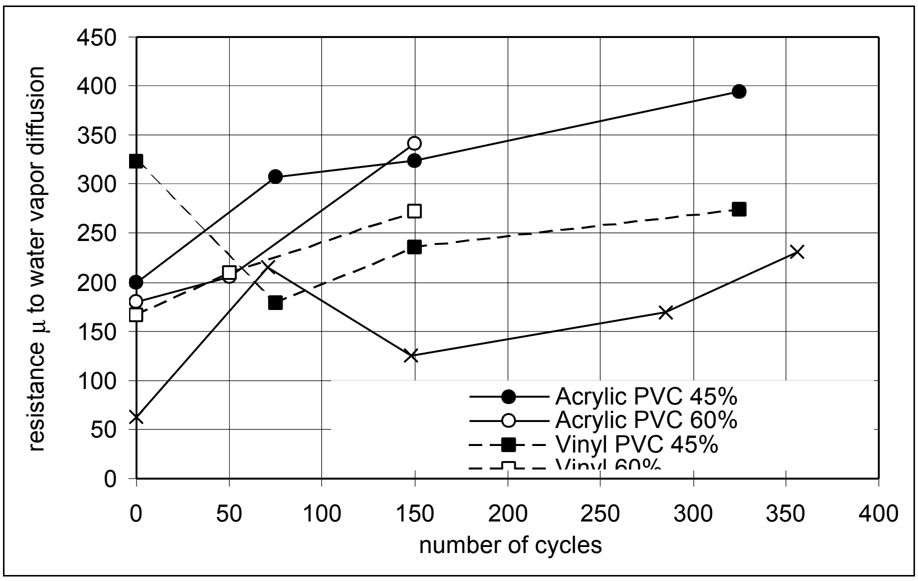


**Figure 4. Resistance to water vapor diffusion as a function of the number of cycles.**

The curves displayed in Fig. 4 show that the presence of the paint results in an important increase of the resistance to water vapor diffusion.

A general increase of resistance is also observed with increasing number of weathering cycles for all types of paint. Acrylic paints show the highest resistance to water vapor diffusion, but differences between paints seem not to be so relevant.

## 5 MICROSTRUCTURE ANALYSIS

The results of microscope's analysis are summarized in the photographs at the end of aging tests presented in Fig. 5 and 6, chosen from the series of photographs done for the different samples, for different aging times, and at different depths (at the external plaster's surface, within the plaster and at the interface between brick and plaster).

From the microscope's analysis we found that the plaster microstructure doesn't show any time variation at different depths for painted samples; on the other hand from surface analysis we could evaluate different degradation effects for the different used paints.

For the vinyl resin's at 45% of Pigment Volume Content (PVC), after 75 cycles we found localized cavities within the protective film and surface blistering; at 150 the cavities expand unto relevant parts of the interface between plaster and the protective film, causing local cracking in this film; finally at 325 cycles we have an evolution of degradation effects unto relevant detachment of the protective film from plaster (Fig. 5).

In the case of the vinyl resin's at 60% of Pigment Volume Content (PVC) at 50 and 150 cycles we obtained similar results as for 45% PVC ending with cracking of the protective film.

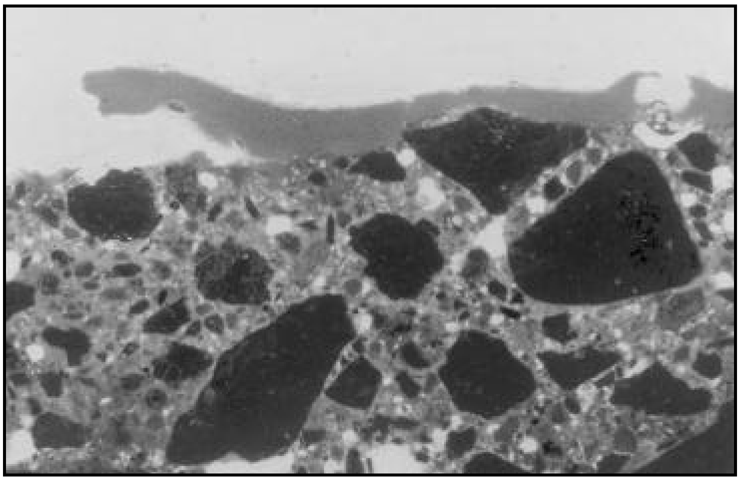

**Figure 5. Microscope's analysis: vinyl resin's paint (PVC 45) after 325 cycles**

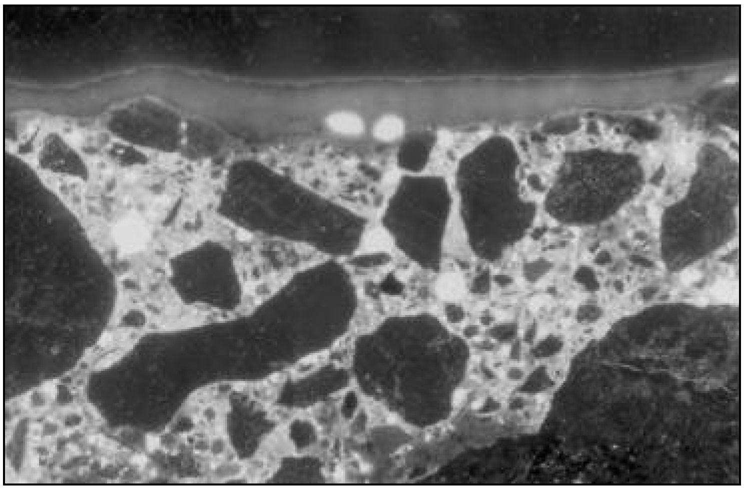

**Figure 6. Microscope's analysis: acrylic resin's paint (PVC 45) after 325 cycles**

On the other hand the acrylic paints show no degradation until 150 cycles: in the case of lower resin's content (60 % of PVC) some cavities appear in the protective layer at the end of aging test while in the case of higher resin's content (45 % of PVC) only a few discontinuities between protective film and plaster are rarely visible: for this paint cavities appear localized later at 325 cycles (Fig. 6).

## 6 SURFACE PHOTOGRAPHS

The surface of protected samples subject to aging tests has been photographed during time; in fig. 7 we can see the result of surface degradation for the vinyl resin's paint at 45 % of PVC after 325 aging cycles: the surface shows an evident damaged condition, with broken blisters and cracks with salt surface crystallization, probably due to water transfer from internal layers of plaster to the external surface.

After the same aging time of 325 cycles we see in Fig. 8 that the sample protected with acrylic resin's paint at 45 % of PVC show an undamaged surface.

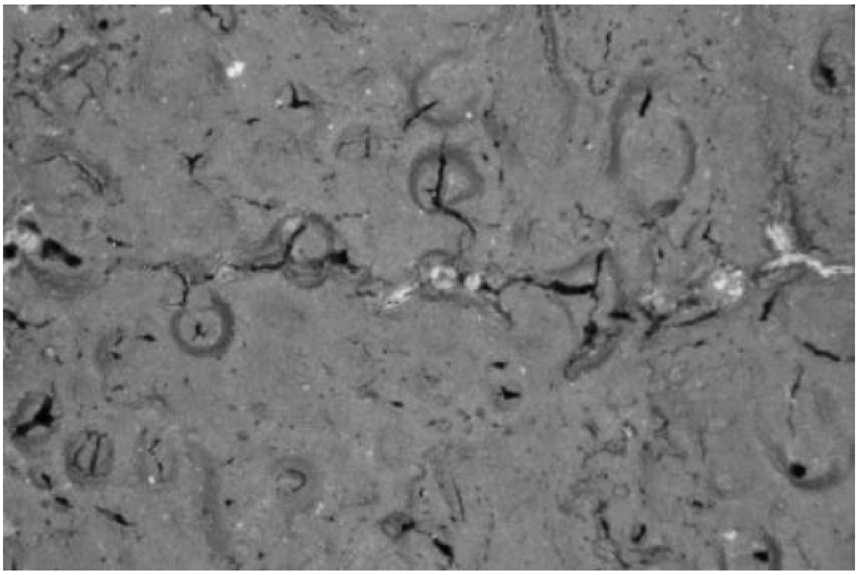

**Figure 7. Surface photographs: vinyl resin's paint (PVC 45) after 325 cycles**

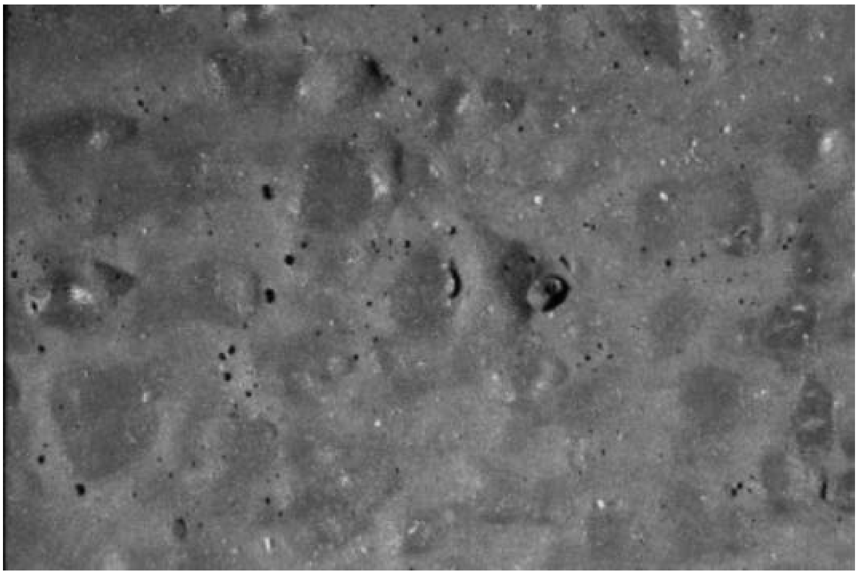

**Figure 8. Surface photographs: acrylic resin's paint (PVC 45) after 325 cycles**

## 7 NATURAL WEATHERING RESULTS

Finally here are presented some first results coming from the outdoor exposure of wall samples (Fig. 9), in Milan and Lugano, as photographs of vinyl resin's painted surface (Fig. 10): after two years we have the same degradation effects with cracks in the protective film as we measured in laboratory aging (Fig. 7).

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**Figure 9. Outdoor natural aging samples**

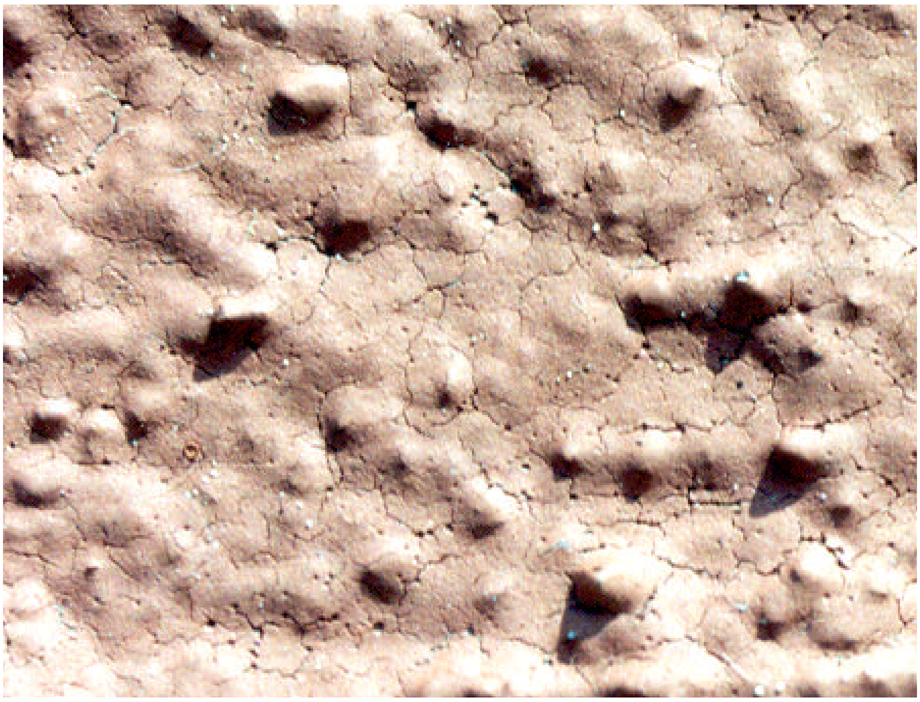

**Figure 10. Surface photographs of outdoor natural aging samples with vinyl resin's paint**

It means that the laboratory aging cycle actually simulates the outdoor conditions, and then it's possible to work on the evaluation of the time re-scaling coefficient, i.e. the ratio between the paints' service life and the number of lab aging cycles.

## 8 CONCLUSIONS

The analysis of the experimental results of this first part of research bring us to the following conclusions:

- The protective degrees to water for the used paints could be evaluated, through the measurement of the mass increase of samples subject to aging cycles (Fig.2) and through the capillary absorption difference between painted samples and plaster (Fig.3);

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  - The acrylic resin's paints have shown a better protective degree to water, compared with the vinyl resin's paints;
  - The higher resins' content allow to obtain a better protective degree;
- During accelerated laboratory aging tests the paints' differences in the time behavior could be evaluated, with the detection of degradation effects, through microscope's structural analysis (Figs 5 and 6) and through surface photographs (Figs 7 and 8)
  - The vinyl resin's paints have shown more relevant degradation effects, after a lower number of aging cycles, when compared with the acrylic resin's paints.

It's evident that these visible damages are linked with the decay of functional and protective characteristics, as the water permeability. Finally we can say that after this first experimental phase we could evaluate the paints' protective degree to water in laboratory through mass increase and capillary absorption. The kind of resin and its content have demonstrated to be the most important parameters to influence the paints' protective degree.

These conclusions are partial as they are regarding only the external layers of the wall component, while the evaluation of component's service life will be obtained at the end of aging tests on the whole wall; actually in the first experimental phase the samples are collocated inside the weathering cell while in the second experimental phase the samples of the whole technical solution will be positioned as the door of the cell, and will be subject to aging cycles simulating the external climate, and to gradients of temperature and vapor pressure between the indoor and outdoor environments, to simulate realistic functioning conditions. Nevertheless the results of this experimental phase demonstrate that the adopted methodology allows to measure the service life of protective paints comparing lab results with outdoor natural aging ones.

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# Building Components' Durability, Maintenance Planning And Sustainability

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**Summary: The paper highlights the importance to know building components durability to operate their choice in compatible terms with regards to their use sustainability as for the differentiated obsolescence of building parts and for a useful building maintenance planning.**

**We point out particularly the reasons why to decide different maintenance typologies to set up the maintenance works planning for different components of the designed elements, depending on their reliability and service life.**

**We propose criteria for the risk probability estimation of early failure that may appear during building components predicted service life, entailing emergency maintenance works, caused by design and/or execution mistakes; this probability may be estimated from values of functional, inherent and critical reliabilities (design pathologies) or from values of component executive reliability (execution pathologies).**

**We conclude highlighting the importance to correlate the choice of building components to their durability (reliability and service life) with the aim to optimize it towards the sustainability of the whole building and of its parts. In fact, if on one side the choice of a building component must at first depend on the fact that it shall supply adequate initial technological performance levels to contribute efficiently, within the building design orchestration, to the fulfillment of the expected environmental conditions, on the other side among different components soundly designed from this point of view, one shall opt for the component that relatively optimizes durability and sustainability.**

**This will need that the chosen component will have a service life functionally and economically coherent with the expected service life of the whole building where it's inserted and that technologies and materials of the elements which constitute the component will supply an acceptable economic and energy saving of the engaged natural and artificial resources.**

**In this balance it should be taken into account also the actual evaluation of the future building parts' substitutions and of the building demolition, estimating the operative potentiality to recycle disused materials.**

**Keywords: components, maintenance, management, sustainability**

## 1 INTRODUCTION

It's important to observe that quality and sustainability are cares that shall permeate the entire decisional process for buildings' design and that they shall be considered at different levels: from territorial planning, to environmental and functional – spatial design of buildings, to technological, executive, operative, management – maintenance and economical design.

Quality and sustainability assume different connotations and peculiarities at this various indicated levels. They constitute steps of the decisional chain and the sequence of decisional acts that follow will be so much valid as the cares taken on quality and sustainability will be adequate and balanced for each step of the chain.

Within the framework of this preamble it's included the phase of building components' technological design, complex and articulated in sub – phases, that assumes relevant importance within the whole building technological design.

It's important to highlight that the distribution of performance loads among the technological subsystems and related technical elements prefigures the behavior and functioning of the building that will be designed and realized, and substantiates the contents of the so-called building technological program of performance specifications.

At first it's needed to observe that the different technological parts of a building shall be designed and realized to contribute adequately to obtain the environmental and functional – spatial quality that the designer has defined as an objective to be pursued.

Here is the complex and crucial question of planning the technological performances' specifications to assign to the building's technical elements, that is the performance loads to distribute to the different building's parts in a way to allow them, as a whole, to give the necessary contribution to obtain the target environmental and functional – spatial quality.

What has been now expressed is referred then to the components' performance specifications planning, that is to the preparatory phase for the technological design of the elements that are intended to be applied to the specific building.

It's to remember that in this planning are to be considered also the durability performances' specifications of technical elements (useful quality of elements), the operative performances' specifications, regarding the economically and operatively acceptable construction (elements' operative quality), and the maintenance performances' specifications, concerning the economically acceptable maintenance activity (elements' maintenance quality).

The components' technological performances' planning constitutes then the input that binds the following phase of technological design, and it's the first step of the decisional chain regarding the technological design of the building and of its parts. This step will be so much "strong" as the cares upon the aspects related to quality and sustainability will taken into account.

## 2 THE BUILDING COMPONENT'S TECHNOLOGICAL DESIGN

The technical elements technological design represents a complex phase because it shall bring to components that achieve a relative optimization for the different aspects of quality that characterize them. These aspects are presented in the following table 1.

First of all the aspect related to the assignment of initial quality (zero time) to the component coherent with the technological performances' specifications.

It implies to face the technical element's design defining the technological functions that must be fulfilled for each technological requirement and for the corresponding performance's specification.

The component must be evaluated also in terms of the estimation of its functional reliability. This evaluation is worked out in particular on the building component's object model and it' based on the examination of the distribution of analytical functions on the different elements of the component; this functions' distribution may imply a major or minor risk of performance loss in relation with the fatigue to which the components' functional elements are subject in service conditions.

The judgments categories are three: the object model's simplicity, the fatigue implied by the model, the distribution of analytical functions on the model's functional elements. It's obvious that the results of this evaluation could bring to feedback on the component's design.

The second quality aspect that must be taken into account for the component's design is related to the assignment of useful quality (quality over time), coherent with the planned durability performances' specifications.

It's to verify that the service life and the reliability over time of the technical element and of the functional elements that constitutes it are compatible with these performances' specifications. We remember that the information about service life is lacking and not so much reliable, and it's for this reason that the experimental research is being developed on this problem; on the other hand a method for the estimation of the reliability over time, which can be worked out by calculation, is already available (Rejna 1995). It allows, for the moment partially, to work out this verification and to control for this aspect the component's sustainability, which is here an important control.

A third aspect of quality that must be taken into account for the component's design is related to the assignment of construction quality (operative quality), coherent with the planned operative performance's specifications. It's to verify the component from the point of view of the executive complexity that it implies.

**Table 1. Evaluation of conventional quality of the building component**

<table>
<tr><td colspan="2">CONVENTIONAL QUALITY:<br>qualitative classification of components through an indicator of performance propensity related to an objective function that for each class of technical elements define it in relation with specific requirements for that class<br>SPECIFIC REQUIREMENTS:<br>behavior requirements necessary for the right development of the expected functions characteristic for the class of technical elements</td></tr>
<tr><td colspan="2">$Q_p$ = conventional quality   $r_{1,k}$ = requirements   $p_{1,k}$= performance levels<br>$$Q_p = Qp(p_1, p_2, ...p_k)$$</td></tr>
<tr><td>PERFORMANCE PROPENSITY COMPONENTS</td><td>REFERENCE REQUIREMENTS</td></tr>
<tr><td>$Q_c$= characteristic technological quality<br>performance quality at zero time, out of system and context</td><td>behavior technological requirements specific for a class of technical elements</td></tr>
<tr><td>$Q_u$= useful technological quality<br>product aptitude to maintain initial characteristic technological performance levels $Q_c$</td><td>technological requirements specific for the time behavior:<br>durability,<br>reliability,<br>service life</td></tr>
<tr><td>$Q_m$= maintenance technological quality<br>product aptitude to obtain specific levels of maintenance</td><td>technological requirements specific for a maintenance propensity:<br>controllability,<br>aptitude to cleaning,<br>reparability,<br>aptitude to substitution.<br><br>Parameters:<br>availability,<br>mean time of reparation</td></tr>
<tr><td>$Q_o$= operative technological quality<br>product aptitude to obtain specific levels of economy and operability in the construction phase</td><td>technological requirements specific for the operative technological quality:<br>economy of product,<br>economy of necessary machinery<br>economy of necessary workmanship<br>economy of necessary controls<br>economy of production</td></tr>
</table>

This verification is based in the evaluation of the possible differences between the technical element's design and the built technical element, taking into account that the construction errors appear substantially during the phase of component positioning and during the phase of assembly of materials, sections, units executed in a different way from the design indications. The risk of these errors is as much higher as the technological – dimensional complexity of the component is higher.

The technological – dimensional complexity can be evaluated on the base of judgment's criteria for the product object and relation complexity. It allows to improve the component's design to increase its operative quality and to control the sustainability for this aspect. It means that one have to take care, during the component's design phase, of the operative instructions for the component's construction and of the safety problems on the building site.

A fourth quality aspect that must be taken into account for the component's design is related to the assignment of maintenance quality (maintenance quality), coherent with the defined maintenance performance's specifications, concerning economically and operatively acceptable maintenance activities. It's to verify the component from the point of view of the so-called availability for the maintenance. This verification is based in the evaluation of the intrinsic availability of the building component which can be calculated as ratio 1/MRT where MRT is the Mean Reparation Time of the component intended as the mean value of the reparation times for each functional layer of the component, regarding the possible failures that may occur.

The methodology to estimate the availability, to find the possible failures and to calculate the Mean Reparation Time is available.

This allows to have a feedback on the technical element's design to increase its availability ant to verify its management sustainability.

A fifth quality aspect that must be taken into account for the component's design is related to the verification of chemical – physical compatibility that characterizes the different materials that constitutes the functional elements of the building component. These kind of incompatibilities compromise during time, in a more or less accelerated way, the structural integrity of the component causing the collapse, and reveal errors in the component's design presaging of early pathologies (design pathologies).

The evaluation of chemical –physical compatibility (critical reliability) is based on the analysis of the degree and number of incompatibilities at the interface between the functional elements of the components and between the products that constitute the functional elements of the component itself. It's available a method to use for this evaluation (Rejna 1995).

The estimation of chemical –physical compatibility of the component allows to have a feedback on the technical element's design to eliminate the design errors on this fundamental aspect and the probable pathologic phenomena, as to bring back the component's design to the necessary level of sustainability.

A last important quality aspect that must be taken into account for the component's design is related to the verification of inherent quality. The inherent quality is regarding the control about the activation of inherent phenomena that appears during the component's service life because of the seasonal thermal and relative humidity variations of the outdoor environment in the component's use conditions.

These variations appear in the different component's functional layers, and if their intensities are very different in the various layers because of the diversity in values of thermal and humidity dilatation coefficients for the constituting materials, the different dimensional variations between layers will generate stresses that can determine cracks at the interfaces between layers and between the component and the connections with other components (joints), because of the binds that can prevent from a free "dimensional gymnastics".

The verification of inherent quality is based on the evaluation of inherent reliability, that uses different parameters and criteria presented in the existing evaluation method (Rejna 1995).

This verification allows to have a feedback on the design of the component and of its connections, with the aim to eliminate the design errors on this aspect and the probable pathologic phenomena (design pathologies), as to bring back the component's design to the necessary level of sustainability.

We have already mentioned the possibility to evaluate the building component's reliability at service life time, that is the "probability" that the component's service life will be like that. For this aspect it's possible to refer to an already available method that our Department has developed.

This method can be applied on the base of a careful examination of the component's design through a functional and technical analysis of the design itself.

These analysis are done for the four aspects of reliability, and exactly for the functional, executive, inherent and critical aspects (Table 2), that is about the balance degree in the distribution of the analytical functions in the functional elements of the component, on the foreseeable conformity degree of the execution of the component to the design intentions, on the uniformity degree of dimensional changes between the functional elements of the component, throughout its practice, due to the stressing context, on the degree of chemical-physical compatibility characterizing different materials which are the functional elements of the component.

Every analysis about the four aspects of reliability is carried on following some right procedural directions that we have to do for these tests.

**Table 2. Evaluation of the reliability at life time of the building component**

| | |
|---|---|
| **$A_F$** | **Functional reliability.** Balance degree in the distribution of the analytical functions in the functional elements of the component estimating through the observation of the functional model of the component. It's an index of the stress to which the component will be subjected in its practice phase. |
| **$A_E$** | **Executive reliability.** Degree of foreseeable conformity of the component execution at the intentions of the project. It's an index of the foreseeable precision in the installation of the component. |
| **$A_I$** | **Inherent reliability.** Degree of uniformity of dimensional changes between the functional elements of the component throughout its performance in comparison with the stress context. It's an index of functional integrity of the component in its practice phase. |
| **$A_C$** | **Critical reliability.** Degree of chemical-physical compatibility which characterizes the different materials making up the functional elements of a component. It's an index of structural integrity of the component. |

The average of four values of reliability is engaged like a tendency to the reliability of the natural durability of the building component. It's a very useful method because it offers the opportunity to estimate the tendency to the reliability of the natural service life of the building component without waiting for so long testing time required by a right estimation of the reliability.

Now knowing the durability (service life and reliability) for building components is fundamental to choose them as compatible with the sustainability of their use as a function of the differentiated obsolescence of building's parts and in particular, for an optimal decision about the maintenance typology to be adopted, to which refer for the maintenance works' planning (Table 3)

**Table 3. Choosing criteria for maintenance typologies**

| **PROGRAMMED MAINTENANCE WORKS** | |
|---|---|
| Maintenance typologies after failure | Components with very high reliability at life time (for not bistable components, i.e. that doesn't pass suddenly from functioning to not functioning situation) |
| Threshold preventive maintenance typologies: for the substitution at a constant age or at constant time steps during service life period | Components with high reliability at life time (for bistable components, i.e. that pass suddenly from functioning to not functioning situation) |
| Maintenance typologies under condition: reparation and substitution maintenance works after inspections, with planned frequency, with the partial or total failure observation | Components with low reliability at life time (for not bistable components) |
| **NOT PROGRAMMABLE MAINTENANCE WORKS (EMERGENCY)** | |
| Emergency maintenance typologies: | Components with: very low functional reliability, very low inherent reliability, very low critical reliability (for bistable and not bistable components) |
| Opportunity maintenance typologies: decision of maintenance works anticipated on other building components in occasion of needs for emergency works on a component or on a component's subsystems | Not bistable components |

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In fact, for equal service life, the estimation of reliability at life time allows to adopt the optimal maintenance typology, and the complement of one of the estimated value guides on the risk of failures which can appear during the component's service life; on the other side the complement of one for the estimated values of the different components of reliability at life time of the building component guides on the risk of failures caused by design pathologies and by pathologies due to the component's on site construction phase.

The occurrence of design pathologies is indicated by the complement of one for the component's functional, inherent and critical reliability values. The occurrence of the component's construction pathologies is indicated by the complement of one for the component's executive (construction) reliability value. We highlight that these pathologies can bring to the need of emergency maintenance works and then not programmable ones.

We conclude observing the importance to correlate the building components' chose to their durability (reliability and service life) with the aim to optimize it with regard to quality and sustainability of building as a whole and in its parts.

In fact if on one side the building component's chose shall depend firstly upon the fact that it should supply adequate levels of initial technological performance to contribute efficiently, within the building design orchestration, to the target environmental conditions fulfillment, on the other side among various components valid from the design point of view, one shall opt for the component which relatively optimizes durability and sustainability.

This requires that the individuated component has a service life functionally and economically coherent with the target one for the entire building where it's inserted and that the technologies and materials of elements which constitute the component will correspond to an acceptable economic and energy consumption of used natural and artificial resources.

In this balance it shall be introduced with the right weight also the actual evaluation of future substitutions of building's parts and of building's demolition, estimating the operative potentiality to recycle the disused materials.

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# Crystallisation Tests On Treated Brick/Stone Masonry Specimens For Damage Evaluation

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**Summary: It is known that water-proof and consolidant surface treatments cannot be carried out in presence of soluble salts due to the possible formation of cryptoefflorescences causing detachment of the treated parts. An European Community (EC) contract was developed aimed to establish the maximum salt content below which the surface treatment do not fail. A large number of crystallisation tests was carried out on single components (bricks and stones) and on masonry wallettes treated with a water repellent and a consolidant. Different types of salt solutions and different salt concentrations were used. The test was carried out according to RILEM recommendations; the long term damage caused by the salts was measured by visual inspection, photographic survey and laser profilometer. The laser device allows for measurements along chosen profiles on the surface masonry: it is so possible to quantify with the variation of the profile depth over time, the loss of surface material. This value has been used as a suitable parameter describing the time and extent of the material deterioration process.**

**Keywords. Durability, Brick and Stone Masonry, Crystallisation test, Surface treatments**

## 1 INTRODUCTION

Applying surface treatments to prevent masonry decay is an easy and common but costly practice which is thought to avoid or reduce masonry damage and hence delay the ordinary maintenance. Research carried out in the past have reached the conclusion that water proof and consolidant surface treatments cannot be carried out in presence of soluble salts due to the possible formation of cryptoefflorescences causing detachment of the treated parts. Nevertheless it is known that some percentage of soluble salts is always present in a masonry, an EC contract was developed in order to establish, in water presence, the maximum salt content below which the surface treatment do not fail. Crystallisation tests, following RILEM TC 127 MS Recommendation, were carried out, at D.I.S, Polytechnic of Milan, on single components (bricks and stones) and on wallettes treated with a water repellent and a consolidant. Different types of salt solutions and different salt concentrations were used.

The chosen test is useful to verify in laboratory the performance of brick(stone)/mortar system, also controlling the behaviour under surface treatments. The aim of the test is also prevention, design and quality control of jointing and pointing, or even re-pointing mortars on a masonry support, reflecting the site situation. The test is carried out according to RILEM recommendations; the long term damage caused by the salts is measured by visual inspection, photographic survey and laser profilometer. From previous research [Binda et al, 1992], it was shown that since the salt damage in a masonry or porous material proceeds from the external surface to the interior, the loss of material measure as a profile damage can be used as a parameter to measure and monitor decay [Baronio et al, 1993]. The laser device allows for surveying chosen profiles on the surface masonry, qualifying with the variation of the profile depth over time, the loss of surface material. This value has been used as a suitable parameter describing the material deterioration process. Usually the durability tests, which require long term and hence high costs, are carried out on single masonry components, brick, stone or mortar [Binda et al., 1985], [Van Hees et a., 1996]. Nevertheless in the last decade it became clear the limit of tests on single materials, when it is necessary to predict the durability of a masonry. It was in fact found by the authors already in 1985 [Binda & Baronio, 1985], that in a wall the mortar has a large influence on the durability of bricks under salt crystallisation decay, according to the porosity characteristics of the two materials. Therefore a crystallisation test on wallettes using as ageing agent a soluble salt was suggested, the test set up by TNO (Delft, NL), was included in the RILEM TC127MS Recommendations [RILEM MS. A.1, 1998].

Within the above mentioned EC Contract the wallette test was carried out on some of the materials proposed by the Italian Partner in order to control whether the threshold value of salt concentration found in a previous part of the research for the single materials could be applied also to masonry walls. According to the EC contract, the salt concentration depends on the capillary moisture content (named CapMC) which is calculated on the basis of the water absorbed by capillary rise in 48 hours. The results of the salt crystallisation tests suggested that, for the treated single substrates, the lowest concentrations chosen

(1% and 2.5% of CapMC) can be considered as threshold value of the salt compatibility. Nevertheless in the equivalent untreated substrates, after 7 months, also the lowest salt concentrations were able to produce small efflorescences.

In order to test whether or not this value can still be considered as a threshold value for masonry the same low salt concentrations were used for the wallettes: 1% and 2.5% of capillary Moisture Content of the single units. $Na_2SO_4$ was used for all the type of masonry specimens while the $MgSO_4$ was used only for softmud bricks and serena sandstone. $Na_2SO_4$ is one of the most tested salts due to the high damage it is causing to porous materials. $MgSO_4$ can also cause high damage and it was used since it is frequently found in Northern Italy damaged walls.

## 2 PROCEDURE OF THE CRYSTALLISATION TEST ON WALLETTES

The test, according to the RILEM Recommendations [RILEM MS A.1., 1998] detects the durability of masonry which is exposed to moisture variations in a wall, in the presence of soluble salts. The long term damage caused by the salts was measured by visual inspection, photographic survey and laser profilometer.

A wallette (250x200x120 mm) is put in contact with its back side with a salt solution at a chosen concentration and then stored over a layer of dry gravel in a plastic container (open at the top) with the upper face exposed to the environment (controlled laboratory environment of 20°C and 50% R.H.). After closing the remaining open spaces of the container with polystyrene foam strips, evaporation can occur only through the upper surface of the wallette and on that surface the decay is surveyed (fig. 1a,b).

Each four weeks the wallettes are subjected to: a) photographic survey; b) cleaning from efflorescences and detached material with soft brush and a vacuum cleaner; c) again photographic survey; d) description of the observed damage, e) reading the surface profiles by means of the laser profilometer allowing for a quantitative measurement of the surface decay. Demineralized water is added after three months, when approximately the wallettes are approaching the constant mass in order to start a new cycle and to accelerate the damage.

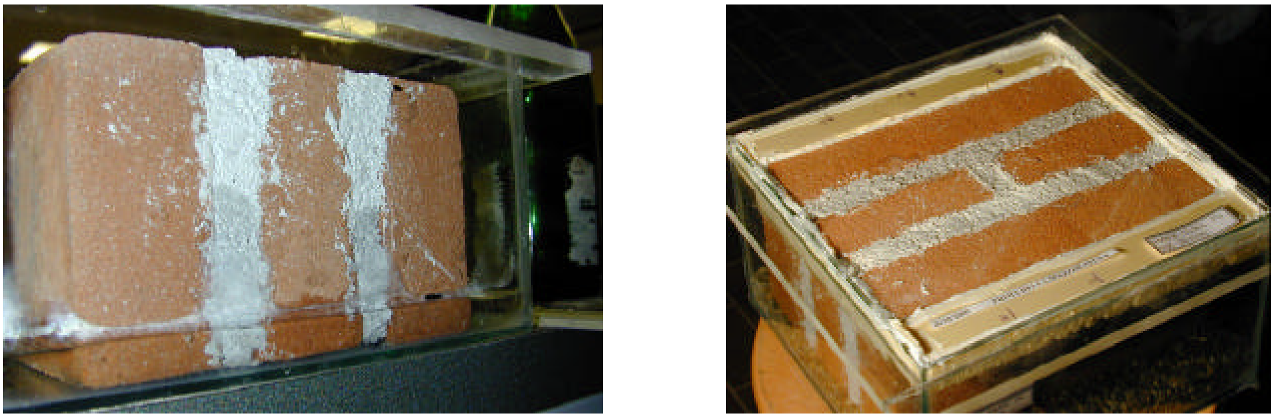


**Figure 1. Preparation of the specimen: a) a wallette is contaminated with salt solution b) the wallette stored on dry gravel into a container and evaporation is allowed from the top surface**

## 3 DAMAGE MEASUREMENT

During their service life, masonry structures, subjected to an aggressive environment, may suffer surface decay according to the type of their components and the construction technique. The great randomness connected with the occurrence of critical attack suggests approaching the deterioration process in a more precise way and with different methods. Deterministic models are yet not available to study the complex behaviour of masonry. Nevertheless once the decay can be measured continuously through a representative parameters, non-deterministic ways can be found to model the decay [Garavaglia et al. 2000a]

### 3.1 Visual and photographical inspection

The chosen test tries to represent the crystallisation phenomenon in a real exposed wall. The formation of efflorescence and sub-efflorescence is connected to migration of salt solutions toward the surface following water evaporation. A brief description of how the wallettes evaporation surface appears and changes over time is important to qualify the type of efflorescence and their localisation (fluffy, bristle, crust, etc.), to define the type of visible damage and to distinguish the effect between the masonry components. In addition it allows for better comprehension of the phenomenon and for controlling constantly the reliability of the quantitative measurements over time. Appropriate photographic documentation is also collected at every inspection.

### 3.2 Laser profilometer

A laser profilometer was used to monitor the damage. The use of the laser profilometer allows for measuring, with a very good resolution, the loss of material from the exposed surface calculated at subsequent times (Fig. 2) [Berra et al. 1993]. Subsequent surveyed profiles show how the surface is changing over time due to the progress of the decay and the loss of material can be measured. Figure 3 shows an example of diagram for the measurements made.

The presence of the typical swelling phenomenon of the surface can compromise the damage measurements. Since bulging is the previous step before detachment, when salt crystallises before the surface, it is possible to consider it as the starting point of a damage. Therefore, through a simple model the experimental measurements have been converted in new deterioration diagrams where the bulging has been eliminated, but considered as a starting point of decay [Garavaglia et al. 2000b]. Single components can also be distinguished to evaluate the different behaviour of the materials when combined. By calculating at each survey the percentage of the area loss of the specimen section, normal to the profile, area loss against time can be represented (Fig.4).

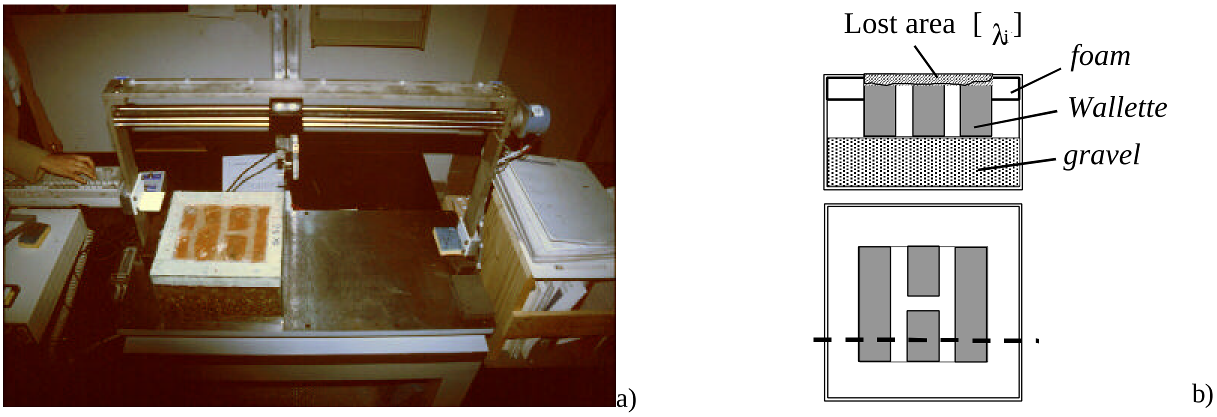



**Figure 2. a) Laser profilometer device and b) scheme of the measurements**

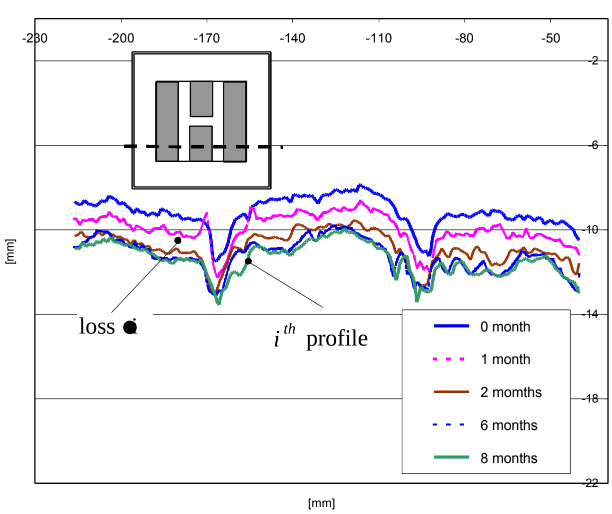



**Figure 3. Example of the first measurements realised with laser profilometer on one wallette**

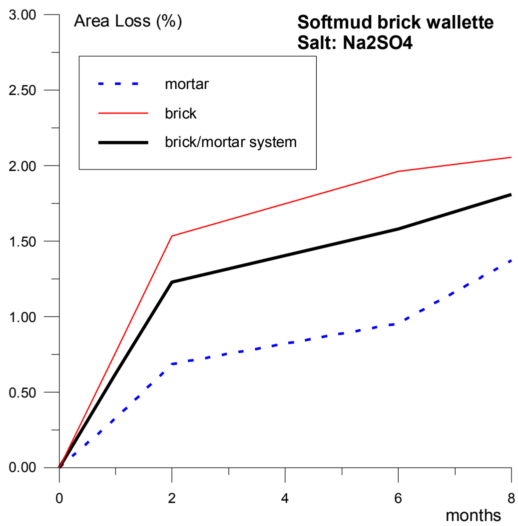



**Figure 4. Deterioration vs. time: example of damage diagram on an untreated brick-wallette**

## 4 MATERIAL PROPERTIES, SALT CONCENTRATION AND TYPE OF TREATMENTS

Crystallisation tests were carried out on one type of softmud contemporary brick used for restoration and on three different natural building stones: (a) Noto limestone, (b) Serena sandstone, (c) Savonnière limestone from France. A physical and mechanical characterisation of the masonry units is reported in Table 1.

All wallettes were realised with bedding joints, 15 mm high, based on putty lime (PLB), except for Savonnière stone wallettes, where a mix of hydrated and hydraulic lime (HHLB) was used. Only in some softmud brick wallettes, a pointing mortar was added (HLP). See Table 2.

In the case of wallettes that were to be pointed, the bedding mortar was left recessed for a depth of 2 cm. Curing conditions: the wallettes after construction were covered for a couple of days with plastic sheets. Then they were moved to a climatic chamber with 20°C and 90% R.H. for 3 months. For wallettes that were to be pointed, pointing was set up after verification of carbonation of the bedding joints and then they were cured at 20°C and 65% of R.H.

**Table 1. Physical characterization of the masonry units**

| | | *Bulk density* (Kg/m$^3$) | *Porosity* (% vol.) | *Capillarity coeff.* (g/cm$^2$/h$^{-0.5}$) | *CapMC* *(%)* | $\sigma_t$ dry | $\sigma_t$ wet |
|---|---|---|---|---|---|---|---|
| Softmud Brick | (SMB) | 1546 | 35.7 | 2.10 | 21.70 | 0.93 | 0.89 |
| Noto limestone | (NO) | 1856 | 29.04 | 0.75 | 12.87 | 2.82 | 1.32 |
| Savonnière limestone | (SA) | 1682 | 18.82 | 0.34 | 9.47 | 1.35 | 0.95 |
| Serena sandstone | (SE) | 2600 | 5.19 | 0.25 | 0.57 | 6.83 | 3.54 |

**Table 2. Classification of the mortar**

| | | |
|---|---|---|
| Bedding mortar: | (PLB) | Putty lime + calcareous sand (max agg. $\phi$ = 4 mm) – Ratio: 1 : 2.7 : 0.2 |
| | (HHLB) | Hydrated lime + hydraulic lime + siliceous sand (max agg. $\phi$ = 4 mm) – Ratio: (0.5 : 0.5) : 3 : 0.8 |
| Pointing mortar: | (HLP) | Hydrated lime + calcareous sand (max agg. $\phi$ = 2 mm) – Ratio: 1 : 3 : 0.8 |

Wallettes were treated, immersing the upper surface into the treatment for 10 seconds for the water repellent and 30 seconds for the consolidant (Table 3). One wallette was left blank for each material to be used as a reference for the decay. Then the wallettes were contamined with salt solution and submitted to the crystallisation test following [RILEM TC 127 MS A.I, 1998]. According to the results achieved with the crystallisation test on single masonry units [Garavaglia et al., 2001], damage was always observed for the highest concentrations tested (5% or 7.5% of Cap.MC.) and in many cases for the lower concentrations (2.5% or 5% of Cap.MC), but never for the minimum (1% of CapMC), which could be considered as a threshold value.

As previously mentioned, it was assumed within the EC contract that the salt concentration depends on the capillary moisture content (CapMC) which is calculated on the basis of the water absorbed by capillary rise in 48 hours. In each specimen an amount of salt solution was introduced, according to [RILEM MS.A.1, 1998] with different salt concentrations for each type of salt: 1% and 2.5% of Cap.MC. These percentages are the lowest salt concentrations used for the single units, chosen also for wallettes in order to verify whether or not these values can still be considered as a threshold value for brick(stone)/mortar composed. $Na_2SO_4$ was used for all the type of masonry specimens while the $MgSO_4$ was used only for a part of softmud bricks and serena sandstone. See Table 3 and 4.

The Water repellent is a solventless silicone microemulsion concentrate based on silanes and siloxanes, diluted with water to yield microemulsion. The Consolidant mainly consists of reactive silicic acid ethyl ester compounds. The material was especially developed to strengthen and consolidate weathered natural stone as well as terracotta, brick, stucco, fresco and loam. The choice of these treatments is suggested by their large use.

**Table 3. Treatments and salts used for the wallettes**

| *Surface* *treatments* | | *Type* *of salts* | |
|---|---|---|---|
| Water repellent | (WR) | $Na_2SO_4$ | (SS) |
| Consolidant | (C) | $MgSO_4$ | (MS) |

**Table 4. Mean value of the salt solution inserted in the wallettes**

| *Wallettes* | *% salt referred to* | |
|---|---|---|
| for all type of units | *1%* | *2.5%* |
| *Wallettes* | *w% of the dry specimen* | |
| Softmud Brick | 0.23 | 0.57 |
| Noto limestone | 0.16 | 0.38 |
| Savonnière limestone | 0.04 | 0.07 |
| Serena sandstone | 0.14 | 0.34 |

## 5 EXPERIMENTAL RESULTS AND DISCUSSION

### 5.1 Softmud brick wallettes

#### 5.1.1 Crystallisation test with $Na_2SO_4$

*a) untreated wallettes*

The damage for the lowest concentration (1%) was very little in the first three months, then it started uniformly in the bricks and in the mortar joints. With the highest concentration (2.5 %) the damage became soon serious due to cryptoefflorescences for both materials (Fig. 5a and Fig.8a,b).

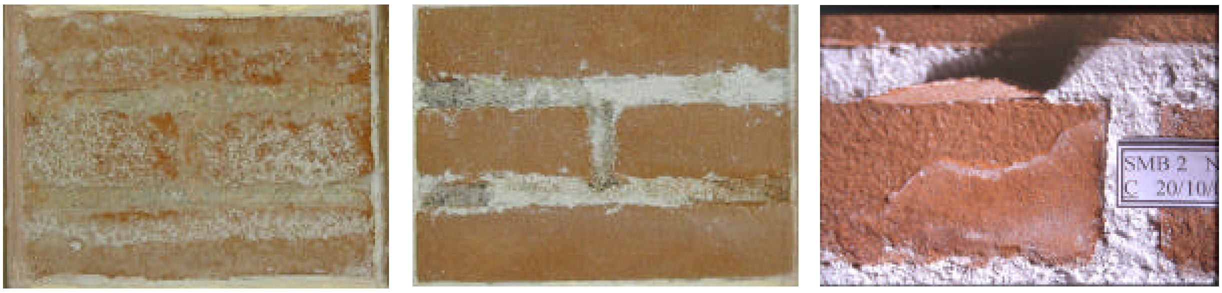
*b) water repellent treatment*

In the first three months both bricks and mortars presented no damage. After three months damage was visible in large part of the mortar joints. The treated bricks with the two salt concentrations do not present damage also in the 8 first months. On the contrary the mortar, as the only vehicle to water evaporation, was damaged over time (Fig.5b and Fig.8a,b).

*c) consolidant treatment*

In presence of consolidant, the damage seems to be, at the beginning, the contrary of what happened to the ones treated with water repellent: the damage started from the brick and then diffused to the mortar. With 1% of salt solution: after 6 weeks not many efflorescences were visible. Then the treated bricks presented soon some scaling of surface layers (2-3 mm), above all in the central bricks surrounded by the mortar (Fig.5c). After 8 months (Fig.8a,b) efflorescences were all over the wallettes and the bricks still have been delaminating (layers of 1,2 mm) and powdering. Mortar did not present particular damage. With 2,5% of salt solution: the treated bricks presented soon cryptoefflorescences starting from the interface brick/mortar and then after 6 months spalling of layers is still continuing. After spalling the treated bricks become sufficiently porous to allow the evaporative flow all over the wallette, without compromising necessarily the mortar, as in the case of water repellent.

As the material loss in untreated wallettes after the same period of time is equal to the ones treated with consolidant, this treatment with this type of bricks seems to be unnecessary to prevent salt crystallisation decay. Up to now no damage is visible on bricks treated with water repellent with both salt concentrations, but the passed experiences suggest to continue the test. In fact, after cutting of the specimen treated with water repellent at 2.5% of salt concentration, a small line of salt was clearly visible under the treated layer (Fig.7a).



Untreated a) Water Repellent b) Consolidant c)

**Figure 5. Softmud brick wallettes with $Na_2SO_4$**

#### 5.1.2 Crystallisation test with $Na_2SO_4$ in presence of Pointing mortar

*a) untreated wallettes*

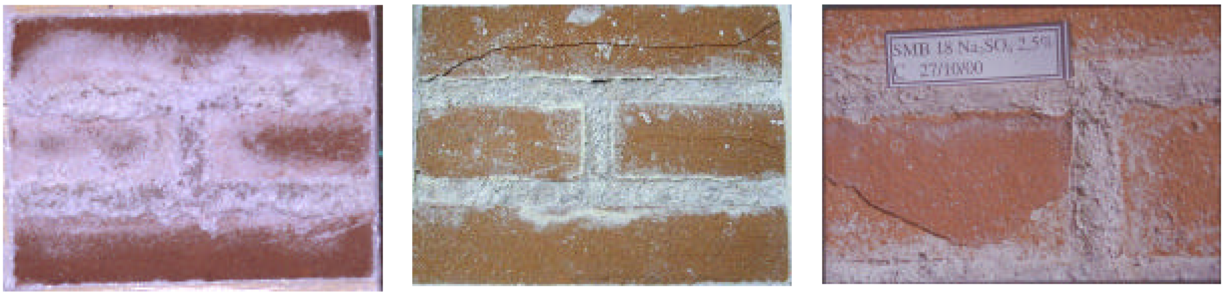
The damage started from the brick/mortar interface and extended to the rest of the bricks causing powdering on all wallette surface for both salt concentrations. The effect is slower compared to the specimens with only bedding mortar (Fig.6a).

*b) water repellent treatment*

As observed in the previous case, in the first months, bricks, treated with water repellent, did not present damage while on the contrary the pointing mortar joints showed serious damage with loss of material > 2mm. However this material loss is less then the one of the bedding joints of the wallettes seen before. After 6 months, the bricks with 2,5% salt concentration suddenly showed a crack on their surface (Fig.6b and Fig.8c,d) and, after cutting, a surface layers detachment of 7-9 mm was observed on all bricks (Fig. 7b and c).

*c) consolidant treatment*

The bricks treated with consolidant, already at the first months, presented detachment of the treated layer, 1,2 mm thick (Fig.6c and Fig.8c,d), and then the bricks became untreated showing uniform powdering. This effect is visible also with the lower salt concentration but more slowly. The pointing mortar presented no visible damage.



Untreated a) Water Repellent b) Consolidant c)

**Figure 6. Softmud brick wallettes with Pointing with $Na_2SO_4$**

a) b) c)

**Figure 7. Softmud brick wallettes, particulars of the damages with WR: a) with 2.5% of $Na_2SO_4$ b) and c) with 2.5% of $Na_2SO_4$ in presence of Pointing**

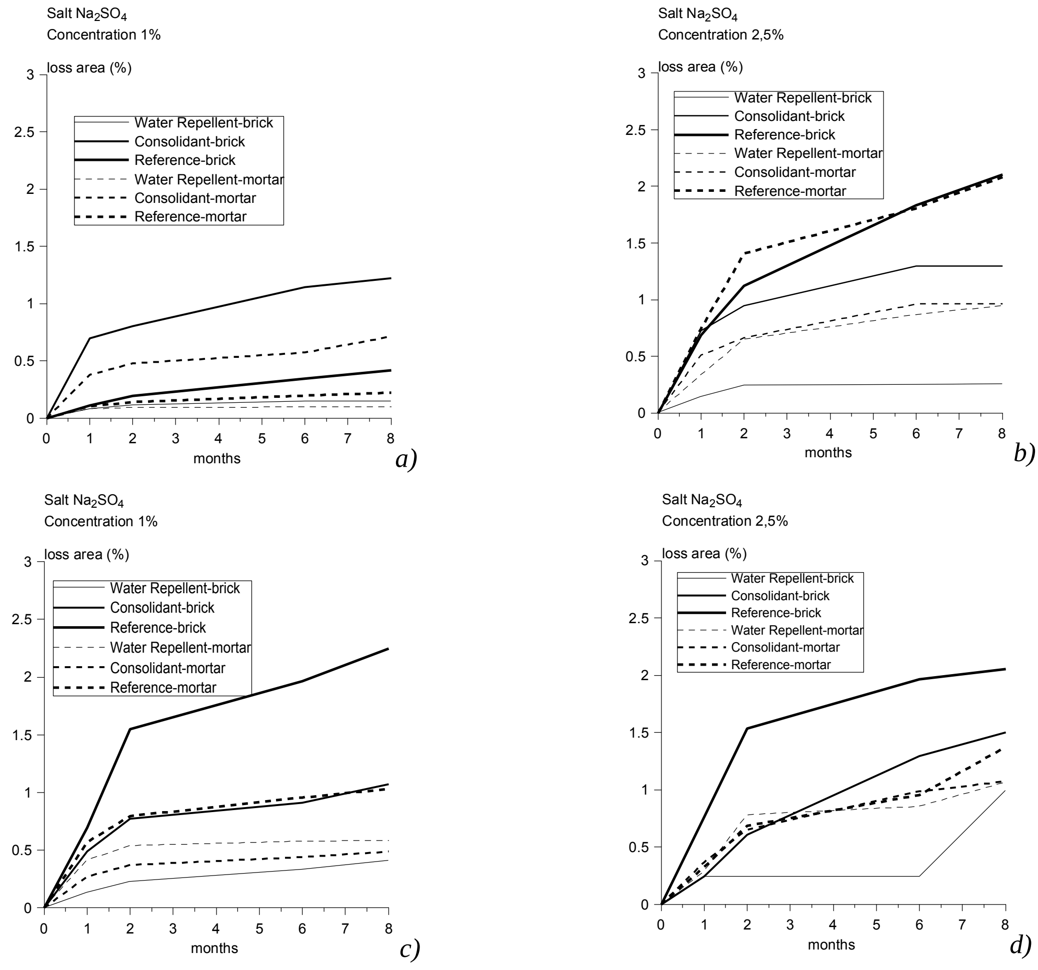



**Figure 8. Deterioration vs. time: damage diagram on a brick-wallettes *a)* and *b)* without pointing and *c)* and *d)* with pointing**

5.1.3 3) Crystallisation test with $MgSO_4$

*a) untreated wallettes*

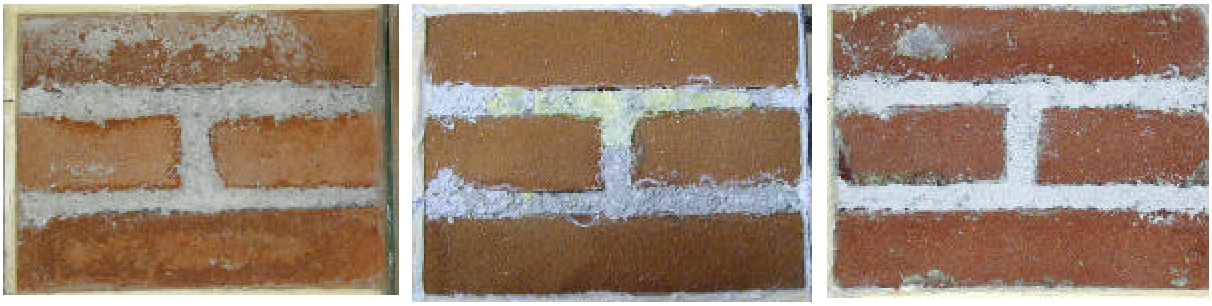
Powdering of the two materials together with hard crusts are visible as in Fig.10. Typical of the $MgSO_4$ is the “wet effect” (Fig.9a).

*b) water repellent treatment*

No damage is visible on bricks and yellow efflorescences are visible on bedding joints (Fig.9b). For the higher concentration some efflorescences are visible also on bricks at the mortar/brick interface.

*c) consolidant treatment*

As observed in wallettes with the sodium sulphate, the consolidant causes more damage in the bricks than in the mortar (Fig.9c).



Untreated a) Water Repellent b) Consolidant c)

**Figure 9. Softmud brick wallettes with $MgSO_4$**

## 5.2 Noto limestone wallettes

5.2.1 Crystallisation test with $Na_2SO_4$

*a) untreated wallettes*

Efflorescences started from the stone/mortar interface but the damage for the lowest concentration (1%) was very poor in the first three months, then it was still concentrated near the mortar joints (Fig.10a). With the highest concentration (2.5 %) the efflorescences were more evident in the stone/mortar interface and the powdering of the stone was increased (Fig.11a,b).

*b) Water repellent treatment*

With the lower concentration the damage started slowly and no particular surface decay was visible on wallettes after the first 3 months. After 6 months the damage due to the presence of water repellent became serious showing exfoliation and spalling of stones (Fig.10b).

With the higher salt concentration the stone damage was serious from the beginning showing spalling of layers of about 1,4 mm. After 8 months the observed damage for both concentration was similar. Mortar joints showed damage with the two concentrations (Fig.11a,b).

*c) Consolidant treatment*

In the short period of 1 month, the stones, also with the lower concentration of salt, presented detachment of a thin layer (about 0,65 mm). See fig.10c. After the removal of the layers the stones started powdering uniformly. The mortar showed damage after 3 months but only with the higher concentration (Fig.11a,b). Measuring the detached layers thickness and the wettability of the stone for water repellent, the depth of the consolidant surface treatment seems to be around 0,65 mm, while the one of the water repellent seems to be 1,4 mm,. The difference in penetration depth could explain the delay in the exfoliation of the water repellent treated surface with the lowest salt concentration and consequently the more irregular damage observed due to water repellent treatment.

As the material loss in untreated wallettes after the same period of time (8 months for the moment) is less or equal to the treated ones, treatments on Noto limestone wallettes seem to be unnecessary to prevent salt crystallisation decay.

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Untreated a) Water Repellent b) Consolidant c)

**Figure 10. Noto stone wallettes with $Na_2SO_4$**

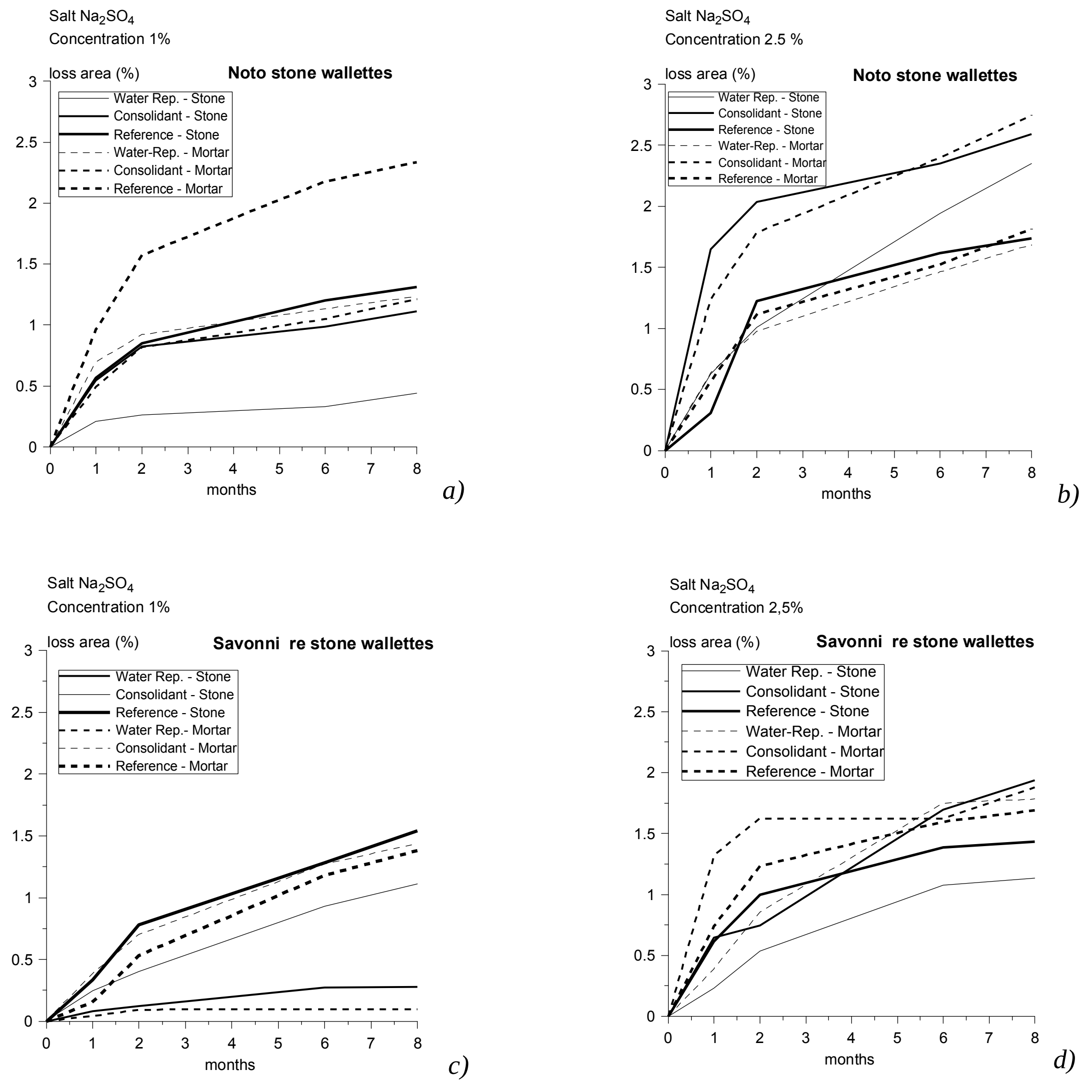



**Figure 11. Deterioration vs. time: damage diagram on a stone-wallettes a) and b) Noto stone and c) and d) Savonnière stone**

### 5.3 Savonnière limestone wallettes

5.3.1 Crystallisation test with $Na_2SO_4$

*a) untreated wallettes*

High dark brown coloured efflorescences and crusts were evident after 3 months (Fig.12a) and the damage consisted in powdering.

*b) water repellent treatment*

For both concentrations at 8 months no efflorescence and damage were visible on stones and mortar (Figs.11a,b and 12b). The mortar is an hydraulic lime mortar, different from the other wallettes.

*c) consolidant treatment*

At three months brown coloured efflorescences and crusts were visible on the wallettes, increased after the second cycle of crystallisation but, although the brown encrustations were still present, the loss of material refered only to a light powdering of both stone and mortar (Figs.11a,b and 12c).



Untreated a) Water Repellent b) Consolidant c)

**Figure 12. Savonnière wallettes with $Na_2SO_4$**

### 5.4 Serena sanstone wallettes

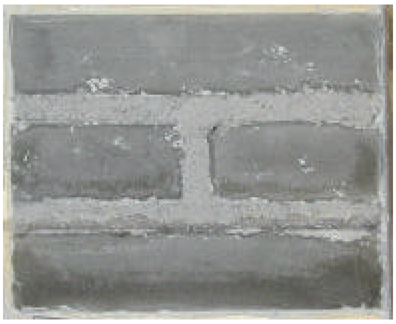
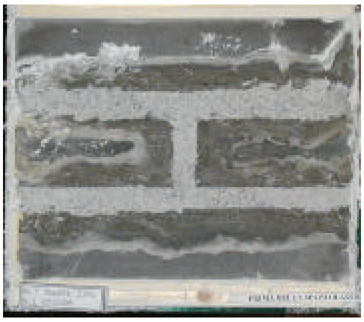
5.4.1 Crystallisation test with $Na_2SO_4$ or $MgSO_4$

*a) untreated wallettes*

Only some efflorescences started to appear in the untreated wallettes at the mortar/stone interface in both case of $Na_2SO_4$ and $MgSO_4$. With the highest concentration of $Na_2SO_4$ was clearly visible the stone surface damage due to salt crystallisation (Fig.13a,b and c).

*b) water repellent and c) consolidant treatment*

At 8 months no particular damage was visible. Only some very small exfoliation are at present visible at the stone/mortar interface.


Untreated with $MgSO_4$ a) Untreated with $Na_2SO_4$ b) and particular of the damage c)

**Figure 13. Serena sandstone wallettes**

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## 6 CONCLUSIONS

Some remarks and conclusions can be given following the research:

- when dealing with durability of masonry, the crystallisation test must be carried out on wallettes rather than on single units;
- the test was useful to calibrate a probabilistic model, which can be of help to evaluate durability and efficiency of surface treatments and to plan maintenance strategies;
- the assumed parameters measured by the laser profilometer turns out to be appropriate to describe the damage evolution in short and long term;
- the salt concentration threshold assumed for consolidant must be lower than 1% of the capillary moisture content, since the damage appears very early;
- apparently no evident damage appears in the case of water repellent, even with the highest tested salt concentration (2.5%), except for Noto stone. Nevertheless a higher damage can appear in a longer time (Fig. 8a);
- the presence of pointing mortar only delays the damage in case of untreated wallettes, while it causes damage acceleration in presence of treatments;
- it would be difficult to apply the threshold values on site, due to the extreme variability of the salts and of their concentration inside a wall. Nevertheless from laboratory research it can be concluded that 1% of capillary moisture content of $Na_2SO_4$ can be considered as a threshold below which the used water based water repellent does not produce damage on all the tested treated materials, except for the Noto limestone;
- Therefore it can be concluded that there is a threshold for each unit and each mortar/unit combination.

## 7 ACKNOWLEDGMENTS

The authors wish to thank E. Garavaglia, the students S. Fara, A. Trittoni, S. Carnazza and the technicians M. Cucchi, M. Iscandri, and P. Perolari. The research was carried out with the EC support (PL970175).

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# Porosity And Diffusivity Of Concrete With Long-Term Compressive Strength Increase Due To Addition Of The Set Accelerator Calcium Nitrate

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**Summary**: **Concrete with additions of the set accelerator calcium nitrate (CN) exhibit a long-term strength increase. Investigations by capillary suction of water and chloride migration showed that the strength gain could not be explained by any decrease in porosity. Water vapor diffusivity obtained by mass loss during drying did not reveal any differences either. Visual inspections by scanning electron microscope indicated that the interface between coarse aggregate and paste may be somewhat denser when CN is added and that calcium hydroxide may be in the form of smaller crystals. Pore water analysis from mature paste showed a substantial increase in alkali concentration when alkali-free CN was added. This could indicate a substitution of bound alkalis in the CSH structure by calcium ions. Thus, improved aggregate-paste interface, CSH and CH morphology and CSH composition could contribute to the observed strength increase.**

**Keywords. Concrete, calcium nitrate, porosity, diffusivity, strength enhancement**

## 1 INTRODUCTION

Calcium nitrate (CN) is used world wide as an admixture for concrete, in particular for applications utilising its properties as set accelerator (Justnes and Nygaard, 1997a). However, CN may be denoted a multifunctional concrete admixture (Justnes and Nygaard, 1999) since it also works as an inhibitor against chloride induced corrosion of rebars (Justnes et al., 1999). Furthermore, CN also enhances the long-term compressive strength of concrete. CN will obviously increase the very early strength due to the earlier start of $C_3S$ hydration and increased maturity at for instance 8 h (Fig.1). However, for concrete (w/c = 0.45 and 4% SF) an increased strength has been observed also at 28 (Fig. 2) and 220 days (Fig. 3) moist curing, in particular for high strength type cement (HSC) compared to ordinary Portland cement (OPC). The strength gain increased until a dosage of 2 % CN of cement weight and flattened out at this level for higher dosages. This paper present the efforts made to explain this unexpected long-term strength increase, since this property should not be utilised in practice before its origin is understood.

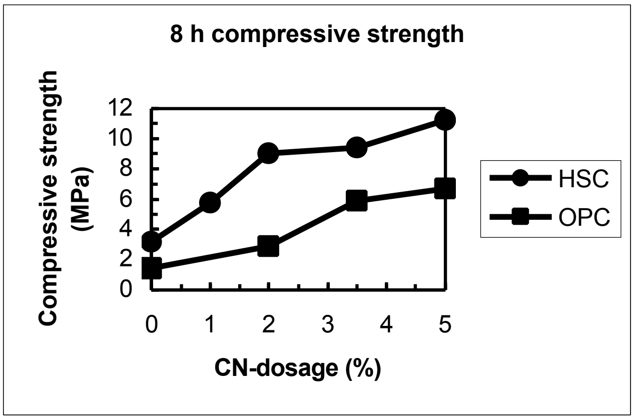


**Fig. 1 Early compressive strength (8 h) for high strength Portland cement (HSC) and ordinary Portland cement (OPC) concrete with different dosages of calcium nitrate (CN).**

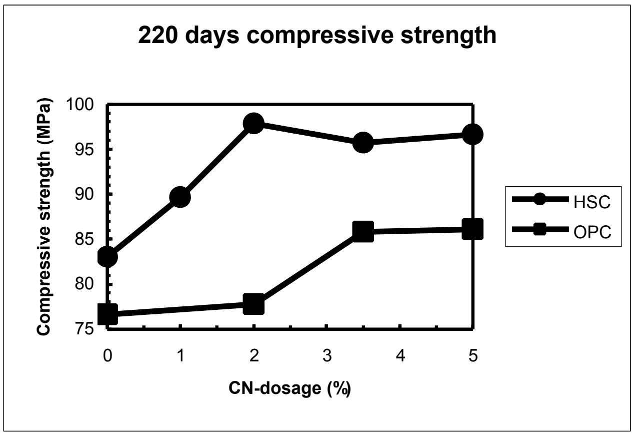


**Fig. 2 Compressive strength at 28 days for HSC and OPC concrete with different dosages of calcium nitrate (CN).**

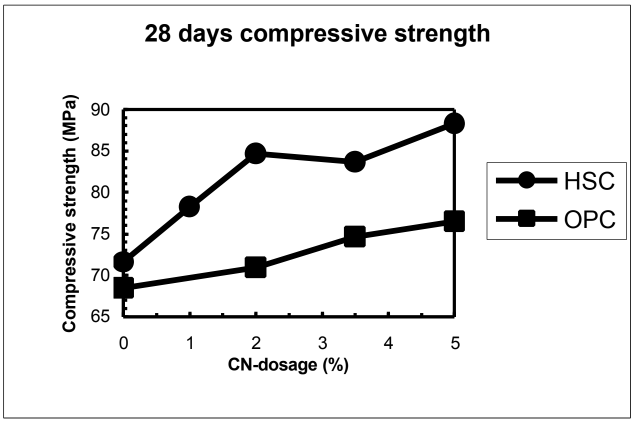


**Fig. 3 Compressive strength at 220 days for HSC and OPC concrete with different dosages of calcium nitrate (CN).**

The first thought was to investigate any change in porosity as a consequence of the CN addition. This was done by capillary suction/saturation by water. Permeability is related to porosity, so chloride migration experiments were carried out as well. Both these parts are also of interest for the discussion of the durability of concrete with CN additions, since it has been common in later years to replace calcium chloride with CN. Additional information was achieved by visual inspection of the microstructure with scanning electron microscopy of plane polished sections, and by studying changes in pore water composition of cement paste with CN additions.

The cement and concrete composition are reported elsewhere (Justnes and Nygaard, 1997a and 1999) but the base recipe (w/(c+s) = 0.45, 4% SF) was of course identical with and without the addition of CN. A granulated, technical quality calcium nitrate (CN) was used that may be expressed as $xNH_4NO_3 \cdot yCa(NO_3)_2 \cdot zH_2O$, with x = 0.092, y = 0.500 and z = 0.826. In other words it consisted of 19.00 % $Ca^{2+}$, 1.57 % $NH_4^+$, 64.68 % $NO_3^-$ and 14.10 % $H_2O$.

## 2 EXPERIMENTAL

Epoxy coated ϕ100 mm·200 mm cylinders of the 220 days old concrete were available for the present study.

### 2.1 Capillary suction and relative water vapor diffusion

Deriving concrete parameters from capillary suction is perhaps not so well known and may deserve a closer presentation: Six 20 mm slices were cut from the middle of each of the epoxy coated concrete cylinders for this experiment. The technique usually requires 105°C drying before capillary suction measurements. Since 105°C may coarsen the porosity, prolonged drying at 50°C was tried out first followed by the normal procedure of 105°C drying. After drying, the discs are placed on a grating 1 mm below the water surface in a covered box. The increase in weight as the specimen suck water is monitored for 5 days and plotted versus square root of time as sketched in Fig. 4.

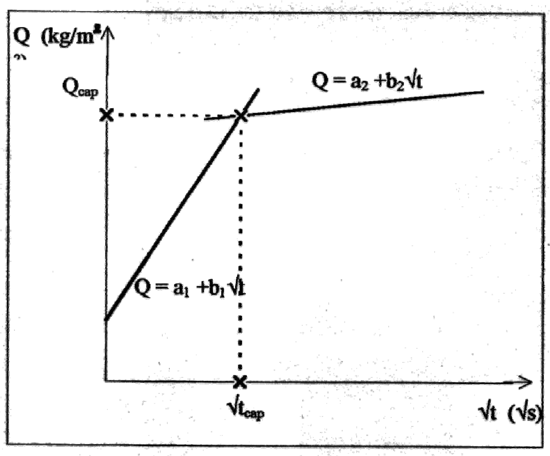


**Fig. 4 Principle of water absorption by a sample in the capillary suction test**

The results from the capillary suction test can be expressed by means of linear regression analysis. Two lines are obtained; the first line represents the value until the waterfront reaches the top of the specimen and the second line the value thereafter. The crossing between the two straight lines is calculated and expressed by the water absorption value ($Q_{cap}$) and time for exposure ($t_{cap}$). These values can be used to calculate the capillary coefficient, k, and the resistance coefficient, m;

$$k = \frac{Q_{cap}}{\sqrt{t_{cap}}} \qquad [\mathrm{kg/m^2 \cdot \sqrt{s}}] \qquad (1)$$

$$m = \frac{t_{cap}}{h^2} \qquad [\mathrm{s/m^2}] \qquad (2)$$

where h is the thickness of the concrete sample (NT Build 368, 1992). Thus, the capillary coefficient, k, gives an estimate of the velocity by which the concrete absorbs water, while the resistance coefficient, m, indicates the time needed for the water front to reach the top.

The procedure consist of 6 important steps for the specimen:

1. Drying to constant weight ($W_1$)
2. Capillary suction for 5 days with weight monitoring
3. Water saturation by submersion 3 days in water at 1 atm ($W_2$)
4. Pressure saturation by submersion 3 days in water at 80 atm ($W_3$)
5. The outer volume (V) is recorded by differential weighing the specimen under water and saturated surface dry in air according to the principle of Archimedes.
6. Drying the specimen to constant weight at 105°C ($W_4$)

From these 6 steps, one can calculate the initial moisture content, total porosity ($\varepsilon_{tot}$), the capillary porosity ($\varepsilon_{cap}$), the air volume ($\varepsilon_{air}$), the average density of the concrete solids ($\rho_{sol}$) and the dry density of the concrete ($\rho_{dry}$) according to the following formulas;

$$Initial \quad moisture = \frac{(W_1 - W_4) \cdot 100\%}{V \cdot \rho_w} \qquad [\mathrm{vol\%}] \qquad (3)$$

$$\varepsilon_{tot} = \frac{(W_3 - W_1) \cdot 100\%}{V \cdot \rho_w} \qquad [\mathrm{vol\%}] \qquad (4)$$

$$\varepsilon_{cap} = \frac{(W_2 - W_1) \cdot 100\%}{V \cdot \rho_w} \qquad [\mathrm{vol\%}] \qquad (5)$$

$$e_{air} = \frac{(W_3 - W_2) \cdot 100\%}{V \cdot r_w} \qquad \text{[vol\%]} \qquad (6)$$

$$r_{sol} = \frac{W_1}{V - \frac{(W_3 - W_1)}{r_w}} = \frac{W_1}{V \cdot \left(1 - e_{tot} / 100\%\right)} \qquad \text{[kg/m}^3\text{]} \qquad (7)$$

$$r_{dry} = \frac{W_1}{V} \qquad \text{[kg/m}^3\text{]} \qquad (8)$$

During the 50°C drying process, the mass versus time to constant weight was monitored as well. And the time, t, to loose 25, 50 and 75 % of the free water was calculated. The diffusion coefficient, D, according to Fick's second law relative to a reference concrete (in this case HSC without CN) can simply be taken as the ratio between the time to obtain the same level of relative free water for the concrete (i) in question and the reference concrete (ref);

$$\frac{D_i}{D_{ref}} = \left(\frac{t_{ref}}{t_i}\right)_{25\%} = \left(\frac{t_{ref}}{t_i}\right)_{50\%} = \left(\frac{t_{ref}}{t_i}\right)_{75\%} \qquad (9)$$

### 2.2 Chloride migration

Another epoxy coated cylinder was cut in 25, 50, 50, 50 and 25 mm, where the 3 middle 50 mm slices were used as parallels for the chloride migration test (NT Build 355, 1989). In this test a $\phi$10mm·50 mm cylindrical specimen is placed between two compartments filled with 3% NaCl solution (compartment 1 with chloride concentration $C_1$) and 0.3 N NaOH solution (compartment 2), respectively. In both solution a metal screen is placed close to the concrete surface and a constant direct current potential of $\Delta U$ = 12 V is applied between them (Fig. 5). The electric resistivity of the specimen is monitored before and after the experiment. Due to the electric field, the chloride ions will migrate through the concrete from the NaCl compartment to the NaOH compartment. The concentration of chloride ion ($C_2$) in the NaOH compartment as a function of time (t) is determined by a spectrophotometric method, and the results plotted like the sketch in Fig. 6.

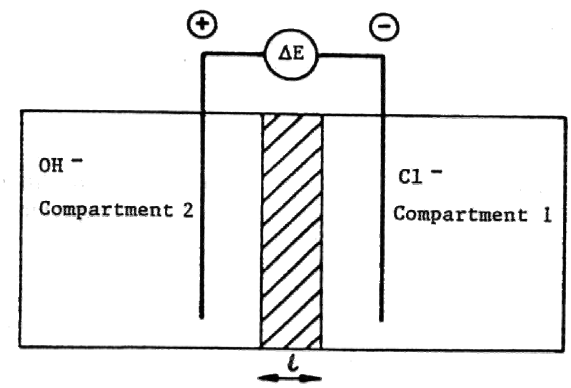


**Fig. 5 Principle set-up of a chloride migration experiment**

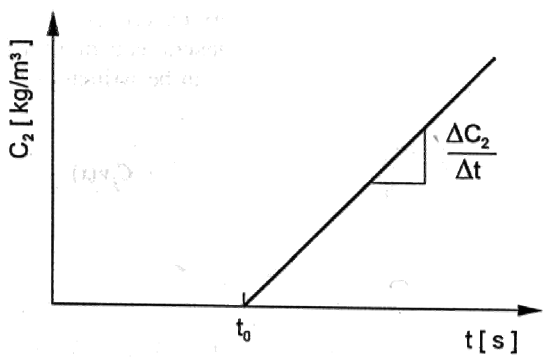


**Fig. 6 Sketch of the chloride concentration in compartment 2 as a function of time**

When a straight line is obtained between the 5 last data points, $\Delta C_2/\Delta t$ is determined as the slope of this line as shown in Fig. 6. Assuming that migration is the dominating transport mechanism and that diffusion and convection can be neglected, the migration coefficient for chlorides can be determined by the formula

$$D_m = \frac{R \cdot T \cdot \lambda \cdot V_2}{z_{Cl} \cdot F \cdot \Delta E \cdot g_{Cl} \cdot C_1 \cdot A} \cdot \frac{\Delta C_2}{\Delta t} \qquad (10)$$

where:

R = the universal gas constant = 8.314 J/mole·K

T = absolute temperature [K]

∝ = thickness of the sample [m]

$V_2$ = volume of compartment 2 [$m^3$]

$Z_{Cl}$ = electrical charge of chloride = -1

F = Faradays number = 96,500 C/equivalent

$\Delta E$ = voltage drop over the specimen [V]

$\gamma_{Cl}$ = activity coefficient of chloride (taken as 1 here for simplicity)

$C_1$ = chloride concentration in compartment 1 [$kg/m^3$] or [mole/l]

A = cross-sectional area of sample [$m^2$]

$\Delta C_2/\Delta t$ = steady state change in chloride concentration versus time in compartment 2 [$kg/m^3 \cdot s$] or [mole/l·s]

It should be noted that the voltage drop across the specimen, $\Delta E$, is less than the applied voltage, $\Delta U$:

$$\Delta U = \Delta E + E^{rev}_{H_2O} + h \qquad (111)$$

where $E^{rev}_{H2O}$ = reversible dissociation potential of water = 0.463 V at 25°C and pH = 13 and $\eta$ = polarisation effects [V]. In this study $\Delta U = \Delta E$ for simplicity.

### 2.3 Other methods

10 mm cubes cut from concrete were impregnated with epoxy under vacuum and plane polished. The microstructure of these specimens was inspected by scanning electron microscopy in the backscattered electron mode.

Finally, pore water was pressed from mature cement paste specimen and analyzed for a number of ions to gain some supplementary information about the microstructure.

## 3 RESULTS AND DISCUSSION

### 3.1 Capillary suction/adsorption and relative diffusion

The results from the capillary suction test for concrete based on high strength cement dried at 50 and 105°C are listed in Tables 1 and 2, respectively, while similar results for concrete based on the ordinary Portland cement dried at 50 and 105°C are given in Tables 3 and 4, respectively.

There is no significant change in capillary porosity within each series with varying CN dosage. The capillary coefficient, k, and the resistance number, m, are relatively constant within each series too. However, the values are larger when the samples are dried at 105°C instead of 50°C due to the drier interior and possible changes in pore connectivity (i.e. coarsening of the pore structure) caused by the more severe drying. There are no significant changes in porosity or dry density that can explain the long-term strength (220 days) gain observed when calcium nitrate is added to HSC concrete.

The diffusion coefficients relative to HSC without CN (ref) according to Eq. 9 were for HSC with 1, 2, 3.5 and 5 % CN 0.96±0.02, 0.87±0,01, 0.88±0.02 and 0.92±0.05, respectively, and for OPC with 0, 2, 3.5 and 5 % CN 0.87±0.09, 0.79±0.09, 0.82±0.06 and 0.84±0.84, respectively. Differences are hardly significant, but marginally lower for samples with CN (lower pore water pressure caused by salts in solution) and for OPC vs HSC (more alkalis; 1.2% vs 0.6%).

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**Table 1. Results with relative standard deviation of the values from the capillary suction/saturation method for HSC concrete as a function of calcium nitrate (CN) dosage when it is pre-dried at 50°C.**

| CN dosage [%] | 0.0 | 1.0 | 2.0 | 3.5 | 5.0 |
|---|---|---|---|---|---|
| Initial moisture [vol%] | 12.3 | 13.0 | 13.6 | 14.4 | 15.6 |
| | ±9% | ±2% | ±1% | ±1% | ±2% |
| $\varepsilon_{tot}$ [vol%] | 14.2 | 14.8 | 15.1 | 15.9 | 15.6 |
| | ±6% | ±5% | ±7% | ±5% | ±6% |
| $\varepsilon_{cap}$ [vol%] | 13.5 | 13.8 | 14.1 | 14.5 | 14.4 |
| | ±6% | ±4% | ±6% | ±5% | ±6% |
| $\varepsilon_{air}$ [vol%] | 0.7 | 1.0 | 1.0 | 1.4 | 1.2 |
| | ±9% | ±16% | ±15% | ±9% | ±13% |
| m [$10^7$s/m$^2$] | 10.4 | 9.6 | 12.0 | 15.0 | 14.3 |
| | ±20% | ±13% | ±39% | ±42% | ±29% |
| k [$10^{-2}$kg/m$^2\sqrt{}$s] | 1.22 | 1.31 | 1.26 | 1.13 | 1.13 |
| | ±13% | ±8% | ±18% | ±16% | ±17% |
| $\rho_{sol}$ [kg/m$^3$] | 2683 | 2687 | 2685 | 2697 | 2703 |
| | ±0.2% | ±0.1% | ±0.3% | ±0.2% | ±0.1% |
| $\rho_{dry}$ [kg/m$^3$] | 2302 | 2291 | 2281 | 2270 | 2282 |
| | ±0.9% | ±0.7% | ±1.2% | ±1.1% | ±1.0% |

**Table 2. Results with relative standard deviation of the values from the capillary suction/saturation method for HSC concrete as a function of calcium nitrate (CN) dosage when it is pre-dried at 105°C.**

| CN dosage [%] | 0.0 | 1.0 | 2.0 | 3.5 | 5.0 |
|---|---|---|---|---|---|
| Initial moisture [vol%] | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| | ±0% | ±0% | ±0% | ±0% | ±0% |
| $\varepsilon_{tot}$ [vol%] | 14.2 | 14.7 | 14.9 | 15.7 | 15.4 |
| | ±6% | ±4% | ±6% | ±5% | ±6% |
| $\varepsilon_{cap}$ [vol%] | 13.2 | 13.3 | 13.5 | 14.0 | 13.8 |
| | ±6% | ±3% | ±6% | ±5% | ±6% |
| $\varepsilon_{air}$ [vol%] | 1.0 | 1.4 | 1.4 | 1.7 | 1.6 |
| | ±8% | ±8% | ±8% | ±6% | ±7% |
| m [$10^7$s/m$^2$] | 6.03 | 5.89 | 5.80 | 5.36 | 6.36 |
| | ±6% | ±7% | ±6% | ±3% | ±18% |
| k [$10^{-2}$kg/m$^2\sqrt{}$s] | 1.59 | 1.66 | 1.74 | 1.78 | 1.62 |
| | ±7% | ±4% | ±7% | ±4% | ±8% |
| $\rho_{sol}$ [kg/m$^3$] | 2682 | 2687 | 2685 | 2693 | 2696 |
| | ±0.2% | ±0.2% | ±0.3% | ±0.3% | ±0.1% |
| $\rho_{dry}$ [kg/m$^3$] | 2302 | 2291 | 2281 | 2270 | 2282 |
| | ±0.9% | ±0.7% | ±1.2% | ±1.1% | ±1.0% |

**Table 3. Results with relative standard deviation of the values from the capillary suction/saturation method for OPC concrete as a function of calcium nitrate (CN) dosage when it is pre-dried at 50°C.**

| CN dosage [%] | 0.0 | 2.0 | 3.5 | 5.0 |
|---|---|---|---|---|
| Initial moisture [vol%] | 13.1 ±1% | 13.4 ±2% | 14.3 ±3% | 14.9 ±6% |
| $\varepsilon_{tot}$ [vol%] | 14.1 ±12% | 14.9 ±11% | 15.6 ±5% | 15.9 ±6% |
| $\varepsilon_{cap}$ [vol%] | 13.0 ±11% | 14.0 ±10% | 14.7 ±4% | 14.9 ±6% |
| $\varepsilon_{air}$ [vol%] | 1.1 ±21% | 0.9 ±18% | 0.9 ±18% | 1.0 ±9% |
| m [$10^7$s/m$^2$] | 14.4 ±16% | 26.0 ±19% | 23.9 ±15% | 17.9 ±14% |
| k [$10^{-2}$kg/m$^2\sqrt{s}$] | 1.02 ±10% | 0.84 ±14% | 0.95 ±4% | 1.03 ±9% |
| $\rho_{sol}$ [kg/m$^3$] | 2688 ±0.3% | 2696 ±0.2% | 2699 ±0.3% | 2700 ±0.3% |
| $\rho_{dry}$ [kg/m$^3$] | 2310 ±2.1% | 2295 ±2.0% | 2275 ±0.8% | 2272 ±1.0% |

**Table 4. Results with relative standard deviation of the values from the capillary suction/saturation method for OPC concrete as a function of calcium nitrate (CN) dosage when it is pre-dried at 105°C**

| CN dosage [%] | 0.0 | 2.0 | 3.5 | 5.0 |
|---|---|---|---|---|
| Initial moisture [vol%] | 0.0 ±0% | 0.0 ±0% | 0.0 ±0% | 0.0 ±0% |
| $\varepsilon_{tot}$ [vol%] | 14.1 ±12% | 14.7 ±11% | 15.6 ±5% | 15.8 ±6% |
| $\varepsilon_{cap}$ [vol%] | 12.9 ±11% | 13.7 ±10% | 14.3 ±4% | 14.5 ±6% |
| $\varepsilon_{air}$ [vol%] | 1.2 ±17% | 1.0 ±19% | 1.3 ±15% | 1.3 ±6% |
| m [$10^7$s/m$^2$] | 6.56 ±7% | 7.62 ±13% | 6.90 ±10% | 6.80 ±18% |
| k [$10^{-2}$kg/m$^2\sqrt{s}$] | 1.51 ±12% | 1.53 ±14% | 1.73 ±7% | 1.64 ±11% |
| $\rho_{sol}$ [kg/m$^3$] | 2691 ±0.4% | 2691 ±0.2% | 2695 ±0.4% | 2702 ±0.4% |
| $\rho_{dry}$ [kg/m$^3$] | 2312 ±2.0% | 2295 ±2.0% | 2275 ±0.8% | 2275 ±1.0% |

### 3.2 Chloride migration

The results from the chloride migration test; the time to chloride penetration ($t_0$), the steady state migration coefficient ($D_m$) and the specific resistance before ($\rho_b$) and after ($\rho_a$) the experiment, are given in Tables 5 and 6 for concrete based on HSC and OPC, respectively, with 0, 2 and 5% calcium nitrate added by weight of cement.

**Table 5. Results with absolute standard deviation from the chloride migration test for concrete based on HSC**

| CN dosage [%] | 0.0 | 2.0 | 5.0 |
|---|---|---|---|
| $t_0$ [h] | 825±30 | 744±39 | 603±18 |
| $D_m$ [$10^{-12}$ m$^2$/s] | 49±20 | 50±1 | 100±25 |
| $\rho_b$ [Ω·m] | 122±12 | 110±12 | 118±5 |
| $\rho_a$ [Ω·m] | 115±4 | 103±19 | 99±3 |

**Table 6. Results with absolute standard deviation from the chloride migration test for concrete based on OPC**

| CN dosage [%] | 0.0 | 2.0 | 5.0 |
|---|---|---|---|
| $t_0$ [h] | 843±27 | 762±84 | 566±24 |
| $D_m$ [$10^{-12}$ $m^2/s$] | 71±19 | 58±16 | 121±13 |
| $\rho_b$ [Ω·m] | 118±22 | 118±20 | 117±8 |
| $\rho_a$ [Ω·m] | 118±26 | 56±27 | - |

There is a tendency for the time to penetration, $t_0$, to decrease with increasing CN dosage, although the change is hardly significant at the 2% CN level. The time to penetration will increase with increasing porosity or pore connectivity and with decreasing chloride binding capacity of the matrix. The steady state migration coefficient, $D_m$, seems to be unchanged by the addition of 2% CN, but increases when 5 % CN is added. $D_m$ should be independent of chloride binding, but increases with increasing porosity and pore connectivity.

The specific resistance, $\rho_b$, before the experiment is apparently independent of the calcium nitrate dosage and cement type. However, there is a tendency for the specific resistance to decrease after the experiment, which can be explained by the increased ionic strength of the pore water due to the sodium chloride content.

There are no observations from the migration experiment that should indicate a decrease in porosity that could explain the long-term strength increase when calcium nitrate is added. On the contrary, the highest dosage seems to increase porosity even though the long-term strength is higher than the reference.

### 3.3 Scanning electron microscopy

The scanning electron microscopy of the HSC and OPC concrete with 0, 2 and 5 % CN additions indicated two trends; 1) the interface between the coarse aggregate and the paste was denser when CN was added and 2) the crystalline mass of calcium hydroxide had a "dotted" structure indicating it to be composed of smaller crystallites. The difference in interface can be observed from the backscattered electron image (BEI) of HSC concrete without CN in Fig. 7 and the HSC concrete with 2 % CN in Fig. 8.

When calcium nitrate is added, the calcium concentration in the water of the fresh mix will be rather high and the hydroxide concentration depressed due to the limited solubility of calcium hydroxide. The $Ca^{2+}$ concentration for HSC paste with w/c = 0.55 without and with 1.55% CN was reported (Justnes and Nygaard, 1997b) to 20 mM and 90 mM, respectively, after 10 min and declining to 16 mM and 52 mM, respectively, after 4 hours (past initial set). At the same time, the hydroxyl concentration was 63 mM and 25 mM after 10 min, and stabilising at 32 mM for both mixes after 4 h. The initial suppression of the calcium hydroxide solubility, could lead to less initial precipitation of calcium hydroxide (CH) on the surface of the coarser aggregate which consequently could mean a tighter interface in the hardened concrete. The suppressed CH solubility could also lead to the formation of smaller crystallites rather than larger CH platelets that could mean weaker zones in the CSH-gel + CH binder. The same applies for any other sparingly soluble calcium salts.

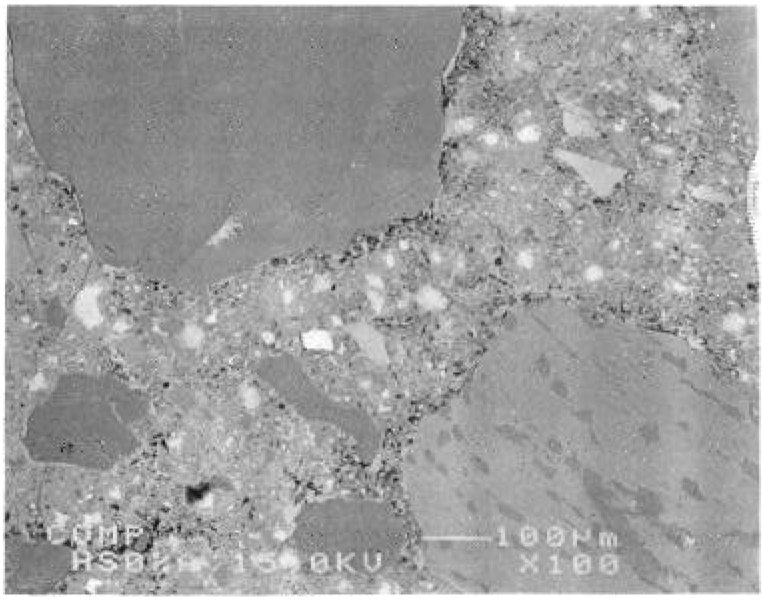


**Fig. 7 100x BEI of HSC concrete without CN**

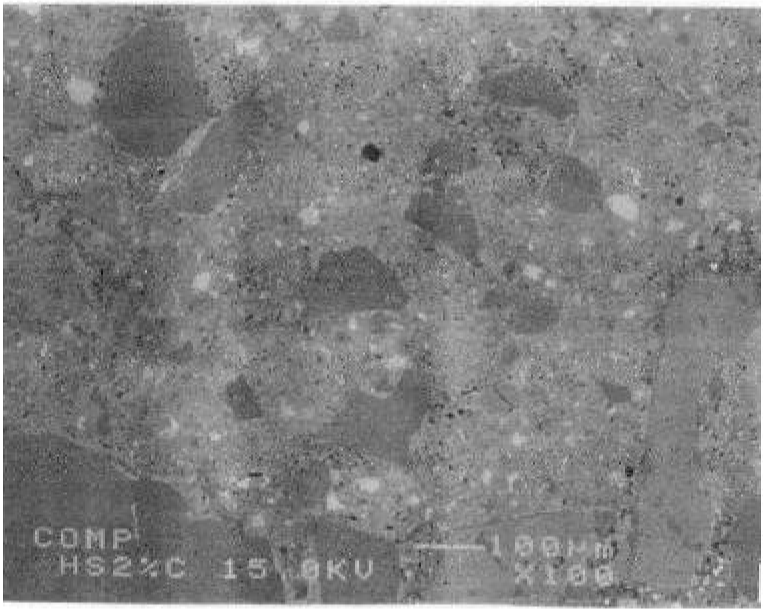


**Fig. 8 100x BEI image of HSC concrete with 2 % CN**

### 3.4 Pore water analyses

The ionic composition of water pressed from mature HSC paste with w/c = 0.55 and without and with 5% CN is listed in Table 7. Even though the CN used was free of alkalis, the alkali content increased substantially when CN was added. This can only be explained by $Ca^{2+}$ replacing $K^+$ and $Na^+$ bound in the CSH gel. Such a substitution can heal dislocations in the CSH structure on an atomic layer and possibly increase its strength. An earlier study of changes in microstructure of paste (Justnes and Nygaard, 1997c) showed by $^{29}Si$ NMR that the average length of the polysilicate anions in the amorphous CSH-gel was prolonged by 17% for OPC and 5 % for SRC (sulphate resistant cement), but unfortunately the HSC was not included in that investigation.

**Table 7. Pore water composition in ppm [mM] and sums of cation ($\Sigma_+$) and anion ($\Sigma_-$) charge of mature HSC paste with w/c = 0.55 and without/with 5% CN.**

| CN-dosage | 0.0% | 5.0% |
|---|---|---|
| $K^+$ (39.10 g/mole) | 690 [17.6] | 9,810 [250.9] |
| $Na^+$ (22.99 g/mole) | 230 [10.0] | 3,390 [147.5] |
| $Ca^{2+}$ (40.08 g/mole) | 70 [1.7] | 210 [5.2] |
| $NO_3^-$ (62.0 g/mole) | 0.5 [0.0] | 16,700 [269.4] |
| $NO_2^-$ (46.00 g/mole) | 2.6 [0.1] | 13 [0.3] |
| $OH^-$ (17.01 g/mole) | - [100] | - [63.1] |
| $SO_4^{2-}$ (96.06 g/mole) | 135 [1.4] | 24 [0.2] |
| $\Sigma_+$ | - [31] | - [409] |
| $\Sigma_-$ | - [103] | - [333] |

Note the excess of negative charge ($\Sigma_-$) in the paste without CN and the excess in positive charge ($\Sigma_+$) in the paste with 5% CN. However, the $OH^-$ concentration is based on pH meter readings to 13.0 and 12.8, respectively, and represents the highest uncertainty. A change in 0.1 pH units in this range corresponds to about 20 mM alone.

## 4 CONCLUSIONS

Concrete with additions of the set accelerator calcium nitrate (CN) exhibit a long-term strength increase. The addition of 2% CN to concrete based on high strength Portland cement, 4 % silica fume and w/(c+s) = 0.45 lead to an increase in 220 days compressive cube strength from 83 to 98 MPa.

Investigations by capillary suction of water, relative water vapour diffusion and chloride migration showed that the strength gain could not be explained by any decrease in porosity. The porosity was rather constant as CN was added, with a marginal porosity increase for the highest CN dosage (5 %).

Visual inspections by scanning electron microscope indicated that the interface between coarse aggregate and paste may be somewhat denser when CN is added and that calcium hydroxide may be in the form of smaller crystals. Morphology changes due to suppressed calcium hydroxide solubility by CN additions due to common ion effects can not be ruled out. Both these effects could contribute to strength increase.

Pore water analysis from mature paste showed a substantial increase in alkali concentration when alkali-free CN was added. This could indicate a substitution of bound alkalis in the CSH structure by calcium ions. Such a healing of CSH dislocations could possibly also strengthen the binder.

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# Technological Complexity And Durability Of Ceramic Suspended Façades

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**Summary: Throughout history architecture of institutional buildings has given importance to walling and envelope systems. Architectural façades were the embodiment of institutional powers, but also played a functional and technical role. Nowadays architectural solutions based on solidity and grandeur are still adopted for institutional buildings, so that durability and appearance are still primary requirements for façade systems.**

**In this ambit the use of double skin ventilated façades has been recently developed as a way to synthetize the traditional aspect and experimented durability of masonry with the modern concept of active walling.**

**Since Renzo Piano's early '80 experiences (IRCAM, Paris) this technology is frequently adopted in Italy and Europe, and recently also in Australia (Aurora Place, Sydney) so that it's now possible to analyse a life cycle of those façades from project to demolition.**

**Excepting for the outer leave, usually made of burnt clay or ceramic, those envelopes had few characteristics in common with traditional burnt clay brick masonry; for this reason it's merely optimistic to forecast for this kind of walls such a long service life. Furthermore some defects of burnt clay, like fragility, can affect the external leave, which can be reduced to 18 mm in thickness. This paper will show an analysis of different cases of suspended façades, from which emerges the importance of a detailed project and the necessity of an accurate choice of materials and maintenance cares: those element concur to individuate some critical points for durability and an estimation of life cycle.**

**It will be shown a general approach on durability assessment looking forward to the substenibility of buildings: previous experiences, like Strathclyde's DAEC, have been experimented in the specific field of building components.**

**Keywords: durability, service life, suspended façades**

## 1 INTRODUCTION

Buildings for offices, and more in general the institutional architecture, symbol of economic and political powers, have faced in the '900 a profound innovation and still represent a vanguard of contemporary architecture. Furthermore in the last twenty years we have seen a marked change of many factors that determine typological and technological characteristics of buildings, as the role of the client and the work organisation. On the other side it is possible to recognise strong elements of continuity that link historical buildings, such as castles and towers, to modern skyscrapers made of steel and glass. The architecture of the building envelope is one of these.

The external layer of contemporary buildings represents a technical and functional system, visa that on these components converge a great number of requirements aimed to determine comfort for users, architectural appearance and cost management in service life. Technical solutions adopted to comprehend this complex of requests at once have carried to the conception of *polyvalent walls.* Those systems have relevant performances regarding thermal insulation, management of solar gain, mass inertia, control of daylight factor, architectural appearance and functional adaptability.

Among these solutions we will analyse the ones equipped with an external layer of ceramic or terra cotta. These double leaf systems, mechanically assembled and suspended to the building structure, generally offer moderate ventilation that enhances energy conservation performances, characterising building architecture with a traditional appearance.

Since Renzo Piano's early '80 experiences (IRCAM, Paris) this technology has been frequently adopted in Italy and Europe, and recently also in Australia (Aurora Place, Sydney). The diffusion of a post-modern language for institutional architecture has definitely encouraged the adoption of these apparently quasi-traditional wall systems.

## 2 ARCHITECTURE AND FUNCTION OF SUSPENDED FAÇADES

### 2.1 A methodological overview

During a research on early stage design decisions for the new building of Pisa's District administration, we have experimented a method to develop and compare alternative solutions for the building envelope (Maffei & Boccaccini 2000), based on the typical approach of value analysis and on decision help tools. The comparison has been made among solutions with similar architectural appearance but relevant differences in performances: the main target was to find critical factors that could justify the adoption of each of these solutions. In this paper it will be shown the analysis and the durability assessment outcomes of that research.

Comparative evaluation of technical solutions in complex applications, such as buildings for institutional use, has been performed within a general framework, based on value analysis, and specific multicriteria tools, particularly Analytic Hierarchy Process (Saaty 1977) and the evaluation tool proposed by prof. Langford named DAEC. The last one provides an assessment framework on sustainability which helps to define the design *brief* requirements, assists the designers to integrate the requirements into the design, and helps the assessors to evaluate different design options. This application provides an evaluation of DAEC potentiality in detailed comparisons of building components.

Others tools, more oriented on building materials sustainability, like NIST's BEES, had been judged useless to determine a complete assessment on durability[i].

Durability requirement represents a crucial issue to obtain quality and value in the construction field: on the other side durability represent a main target for building sustainability, so needs to be studied with a synthetic and global approach. The sustainability concept is now well defined by various researches performed in the '90 (CIB Agenda 21 is one of the main products of this activity). Thinking of sustainability in design decisions and in technical choices involves a multidisciplinary approach, which is necessary to integrate all the analytical appearances in a vision that esteems the attainment of this strategic objective.

Durability needs to be considered as a basic requirement already in the brief phase to guide the choice of technical systems: this approach requires two principal elements:

- a general framework, managed in a group coordinated activity, aimed to homogenise qualitative and quantitative evaluations and to produce judgements and criteria in a scientific and objective way;
- a set of tools, for the scientific elaboration of judgements and criteria.

In this work the problem has been faced with a general method developed from the value analysis one (Maffei 1999).

Value analysis - VA - (UNI EN 1325-1) is an interdisciplinary methodology that, when employed during the planning of a product, process or service, has the target of verifying the functional effectiveness of the proposed solutions compared with the resources and the global costs. The VA team operates following a precise succession of phases: informative, creative, analytical-selective, development and presentation. This approach allows to apply the scientific method to the building design, and in the meantime to co-ordinate an interdisciplinary group activity.

### 2.2 An hypothesis: architecture, first

Without doubt the first reason that stimulates famous architects and common designers to the adoption of suspended façades with external leave in ceramic or terra cotta should be found in the actual architectural tendencies. Many architects, from a side, adopt a post-modern language, in wich the use of traditional materials has a meaning of reconnection with the constructive historical tradition and, often, the regional, vernacular architecture. From the other side suspended façades, with double leaf, thin ceramic tile façades, sometimes ventilated, use old materials and traditional appearance in a new way, modern and experimental. Traditional materials are pushed to new uses and to complex technical behaviours for which they aren't designed for. In this sense the use of terracotta is emblematic.

Burnt clay bricks were used by Romans to build walls and other structures: this material was not adopted for it's structural performances, but only to obtain a finishing layer covering the structural concrete core of the wall. It's only during the middle age that bricks were used primarily for their static contribution in constructions. In the '900 we have seen a progressive abandonment of structural brick masonry in favour of non-structural solutions. Those of them that adopt terra cotta as an architectonic character refer to a great deal of vertical closure types; all of them must primarily assure indoor comfort for building's users and formal characterisation (Acocella 1989).

Burnt clay masonry is still used as external closure for it's proven durability and its capacity to keep the architectonic appearance in time. Structural capacity, resistance to weather - freezing in particular - and pollutant agents make it a very competitive material. On the other side it is evident that the use made nowadays of terra cotta is very different from the original one, and that clay has been deeply transformed to increase his versatility. Innovations have been made introducing additives, new building products and new assembling systems (chemical and mechanical).

Sometimes burnt clay is reduced to a simple choreographic element, especially in façades: producers have stimulated this tendency trying to change all characters (dimensional irregularity, porosity, variability in chemical composition and in colour, etc.) that once were the principal nature and strength of this material.

In this sense the comparison with ceramic and other similar products appears meaningful, since these materials seem better performing in mechanical characteristics and, therefore, more theoretically proper to be used in thin façades.

Eight technical solutions of double leaf external walls, characterised by the use in façade of terra cotta or ceramic elements, have been compared to show the role of these systems in actual user's needs satisfaction, and particularly in the analysis of durability. This approach gave us criteria to correctly operate in the design of such systems and provides us a set of tools to verify design decision.

The relief and the analysis of the real behaviour in time of the examined technical solutions had been made also by means of post occupancy evaluation. This approach can give relevant data for a correct activity of design, if above all the evaluation is made in contexts analogous to those where the project is planned (Kirk & Spreckelmeyer 1993). Technical mistakes, influence of environmental factors, troubles in functionality and defects in construction are the principal elements to underline: although is not possible to generalise these conclusions, relieved defects could be taken under control in each future project.

## 3 COMPARISON AMONG SOLUTIONS

Eight types of technical solutions for external closure have been compared: all of them are characterised by an outer architectonic layer in terra cotta style, an insulation provided by an air layer and insulating pannels, a structural leave made of blocks or a concrete wall. All of these solutions can be classified as (moderately to strongly) ventilated façades.

Ventilated wall is defined by technical standards (like in UNI 8368/2) as a constructive solution of vertical wall characterised by an inner layer of ventilation. This definition doesn't make distinction on the role played by this air layer: we could distinguish two boundary cases:

- natural or forced ventilation of the inner air layer
- igrometrical regulation

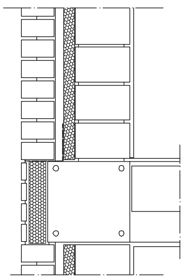
In the first case the external leave is equipped with a series of technical elements (holes, grids) that act to create laminar fluxes of the air, favouring the evacuation of the heat produced by sun radiation;

the second kind of walls uses the air layer, weakly ventilated, to remove the interstitial condense produced inside the wall, usually in winter, caused by the great thermal difference among the outer face of the insulating pannel and the air.

Critical points of these systems are related to their technological complexity: this makes difficult to construct them correctly and maintain them. This complexity is often a discriminatory factor for their adoption, even if ventilated walls can induce a significant reduction in energy cost for the building heating and cooling.

The technical design complexity, especially in joints, corners, intersections with pillars and beams, often resolved by designers with excessive simplifications of details, constitutes one of the weakest points in the architecture of those systems (Acocella 1989).

**Table 1. Description of the compared solutions**

| | | | |
|---|---|---|---|
| Type 1 |  |  | *Double leaf wall with insulating pannel in the cavity.*<br>*Section drawing of a node and picture of a facing brick wall with white spots of salts produced by rain; this wall has no flashing.* |

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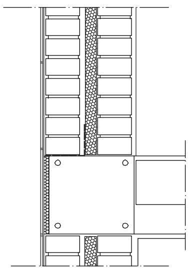
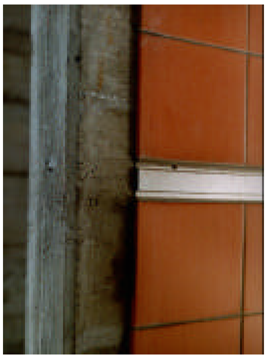
Type 2

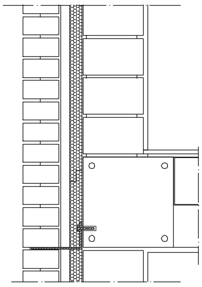


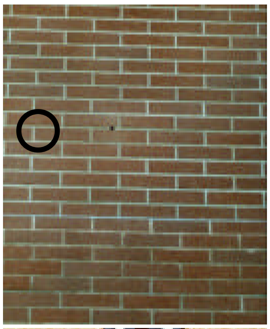


*Double leaf wall with external façade covered with terra cotta tiles, with insulating pannel in the cavity.*

*Section drawing of a node and picture of tiles covering made on a double leaf wall in Versilia's hospital (Camaiore, Italy).*

Type 3

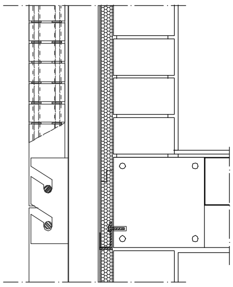


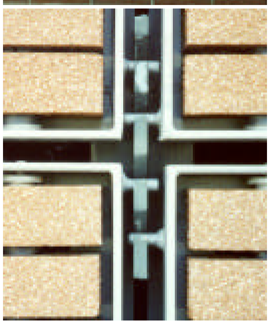


*Double leaf wall with façade in burnt clay bricks hooked by ties to the inner leave.*

*Section drawing of a node and picture of the facing brick façade. You can notice within the black circle a hole for damp evacuation.*

Type 4

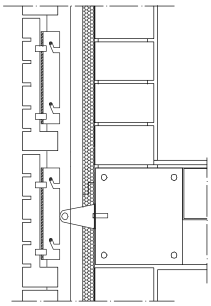




*Double leaf wall with façade in burnt clay hollow bricks mechanically fixed to a metallic structure.*

*Section drawing of a node and detail picture of Colombiadi's node among 4 suspended pannels and the steel substructure (in the middle).*

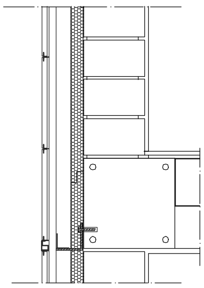
Type 5

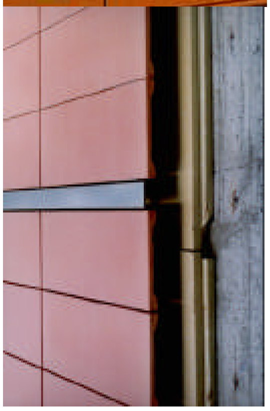


*Double leaf wall with façade in burnt clay pannels mechanically fixed to a metallic structure.*

*Section drawing of a node and picture of Piano's Banca Popolare di Lodi facing terra cotta tiles.*

Type 6





*Double leaf wall with façade in terra cotta tiles mechanically fixed to a metallic structure.*

*Section drawing of a node and picture of Versilia's hospital ventilated façade made in terra cotta tiles.*

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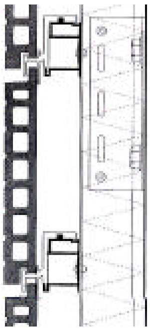
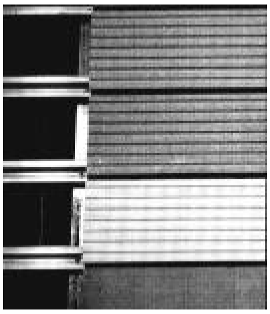
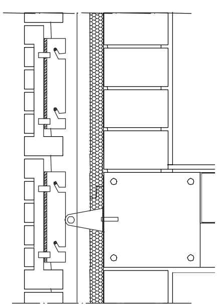
| | | | |
|---|---|---|---|
| Type 7 |  |  | *Double leaf wall with façade in ceramic pannels mechanically fixed to a metallic structure.*<br><br>*Section drawing of a node and picture of Hamburg's Deutsche Messe offices tower designed by Thomas Herzog.*<br>*Façade is made with large ceramic tiles suspended on a metallic frame* |
| Type 8 |  |  | *Double leaf wall with façade in ceramic pannels mechanically fixed to a metallic frame.*<br><br>*Section drawing of a node and picture of the basement of Sydney's Lend Lease office tower designed by R. Piano B.W.*<br>*Façade is made with ceramic tiles similar to terra cotta ones that Piano chose in Banca Popolare di Lodi.* |

### *3.1.1* Type 1. Double leaf wall with insulating pannel in the cavity.

This solution is rather diffused: it is born to protect the insulation layer from unfiltered rainwater. A damp excess can cause an increase of thermal dispersions. The long-term effects can bring to a progressive deterioration of the insulating pannels, especially in case of hygroscopiacal materials (Mor 1996). The cavity, provided with flashing at the bottom (see Table 1), allows the diffusion of damp and its evacuation. This solution is particularly recommended in case of masonry exposed to beating rain, to avoid a rapid degradation (Raffellini *et. al.*1996).

### 3.1.2 Type 2: double leaf wall with external façade covered with burnt clay tiles, with insulating pannel in the cavity.

Wall is made with burnt clay or light concrete hollow blocks, while an external layer provides the requisite of appearance by the use of terra cotta tiles.

Solutions type 1 and 2 are often designed without care of the problematic evacuation of water. This can happen on buildings with small eave length, where it's possible to find some salt deposits produced from the evaporation of the damp accumulated in the burnt clay external leave.

### 3.1.3 Type 3: Double leaf wall with façade in burnt clay bricks hooked by ties.

Facing bricks masonry compose the external leave and steel ties hook it to the inner block wall across the cavity; ties must assure the necessary stability to the horizontal loads. The cavity contains an insulating pannel and can also be naturally ventilated if holes (see Table 1) are provided at the bottom and at the top of the wall. If holes aren't present the cavity has the only function of keeping water away from the inner layers.

Non-structural walls of type 3 often presents fractures because of the inadequate number of ties, or the excessive rigidity of them. In this last case tie is deformed by wall yielding but the reactions it made on the external leave are too strong, so that it can force the bed joints of the bricks.

Steel tie can also be corroded, and this can produce local lesions and breaches in mortar joints.

### 3.1.4 Type 4: Double leaf wall with façade in burnt clay hollow bricks mechanically fixed to a metallic structure.

This solution has been applied in 1987 by Renzo Piano in the IRCAM widening in Paris, and then in 1992 Genoa Colombiadi's exposure building. It is characterised by the external leave, assembled without mortar: a metal bar inserted within brick's holes carry their weight and is then suspended on an upper beam. The open joints among the pannels (see Table 1) make the inner cavity moderately ventilated.

Solution 4 present a similar behaviour in service life in his application in IRCAM as in Colombiadi's building. In this last case it's possible to find fractures and broken elements, probably caused by the reaction among bricks enhanced by the absence of mortar within. Even where bricks are not defective it's often possible to notice a deformation in bricks joints.

### 3.1.5 Type 5: Double leaf wall with façade in burnt clay pannels mechanically fixed to a metallic structure.

A similar solution has been recently used by Piano in the Banca Popolare di Lodi building. joints among pannels are opened and this makes the cavity moderately ventilated.

Comparing this last solution with the previous solution 4 we notice that the greater dimension of external tiles and the wider regulations for the substructure could reduce risks of imperfections in the construction and more in general in the use and maintenance.

### 3.1.6 Type 6: Double leaf wall with façade in burnt clay tiles mechanically fixed to a metallic structure.

This solution has recently been adopted in the new Versilia hospital in Camaiore (Italy). Although the joints among tiles are partially open the inside cavity of the wall can perform natural ventilation.

Solution 6 is fit to realise an efficient ventilated double leaf wall, also because the small mass of the external layer can be easily heated by solar radiation also in winter conditions. On the other side the extreme thickness of those tiles makes them too little resistant to accidental hits: this disadvantage often occur during construction works. To avoid this problem is important to don't assemble the lower part of the façade until other works are completed, and to use a more resistant kind of tile for the first 1-2 meters of wall.

### 3.1.7 Type 7: Double leaf wall with façade in ceramic pannels mechanically fixed to a metallic structure.

In it's Hamburg tower for offices Thomas Herzog tries to furnish an exhaustive answer to the demands of environmental comfort for users, of energetic savings for the client and of sustainability for the whole community. The building envelope is a double wall, equipped with multiple layers, able to transpire and regulate in natural ways the thermal flows among internal spaces and external environment. Plumbing services and cables have been conceived like a vascular net that reaches all the vital parts of the building. Services towers are made with a double leaf suspended wall: the outer layer is made of large ceramic pannels on a metallic substructure.

A problem for durability of all the large pannels suspended façade concerns the joint among pannels and substructure: wind pressure can produce fractures in pannels with linear joints. A large number of evidences[ii] induce to refuse the use of linear joints in favour of punctual ones (Grassi 2000).

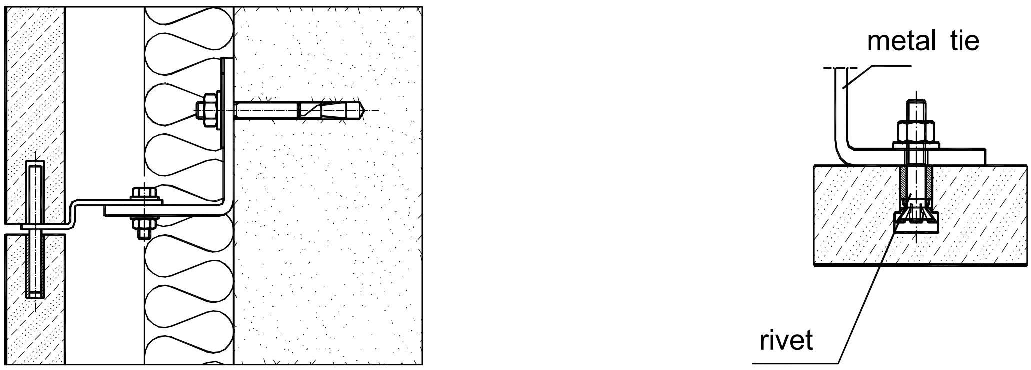



**Figure 1: linear joints (left) and punctual joints with rivets (right)**

### 3.1.8 Type 8: Double leaf wall with façade in ceramic pannels mechanically fixed to a metallic structure.

In the business centre of Sydney, where the need to exploit the available surfaces has carried to the diffusion of tall towers for offices, Renzo Piano Building Workshop's Aurora Place distinguishes itself for the elegance of the shape and for the characterised appearance of the façade. The building has been conceived to fully satisfy the demands of work places spaces expressed by the client, Lend Lease, with the maximum care to the global cost in the life cycle. The materiality of the envelope is realised with ceramic pannels similar to terra cotta facing bricks. This material is used in both the basement of the building as in the inner spaces assigned for institutional and representative activities.

## 4 DURABILITY ASSESSMENT

An analysis of durability and adaptability of the selected solutions of building façade can be developed starting from the individuation of a set of influence factors. These can be reduced to:

- durability of building elements,
- technological and functional complexity of building systems
- critical compatibility of materials;

the last two induce a risk of unsatisfactory behaviour in the service life of the system because of technological and functional incompatibility, while the third factor implicates reduction of components service life because of mechanical or chemical incapability. If a building system is correctly designed and assembled, the theoretical durability of the whole component can be esteemed equivalent to the shortest among those of the building elements adopted. This value should be reduced to take account of a statistic life cycle of construction elements.

In our cases we found that the adoption of ties or substructure in galvanised steel, which has a statistical life cycle of approx. 30 years, is one of the main endogenal risks for those systems.

Decision help tools have been applied to produce a realistic assessment on a specific case study, the brief phase of the Pisa's District administrative building, a 20.000 sq.mt multi-storey centre[iii].

A first qualitative evaluation of durability for the previously listed systems has been produced using in the case study the quoted DAEC evaluation tool, software developed by the Department of Civil Engineering of the University of Strathclyde[iv].

The assessment has been done varying only the façade system, while the rest of building components was defined: the building envelope is a meaningful variable for durability of this building because it should have a large external surface to guarantee natural light and ventilation.

This durability and adaptability assessment of façade systems shows a relevant preference for solutions with ventilated double leaf envelope, particularly because those systems allow an easier maintenance in the life cycle and a great degree of adaptability to new services (see Table 2).

We noticed that DAEC, originally conceived to introduce a general approach to the problem of the sustainability of a built object, can be used also for a deeper evaluation of global durability performances of building elements. To perform this kind of analysis is therefore necessary to modify the given set of criteria, as each element must be described with different relevant performances.

It's easy to understand that, even if  the complete set of requirements and specification for each element is very large, it's possible to find a subset that contains only the requirements derived from the priority functions of the element. This subset changes for each element; i.e., while an external wall is defined mainly with aesthetic, stability, durability and energy specifications, a foundation must be described by stability and durability performances. Requirements that characterise a building component  concur to define the quality of the whole building, so must be strictly planned and verified.

**Table 2: early stage design durability assessment**

| N° of solution | Technical description | DAEC assessment for durability | AHP priority type | AHP priority score |
|---|---|---|---|---|
| **type 1,2,3** | Double leaf walls with facing bricks and insulating pannel in the cavity | **57%** | **1** | **55,6** |
| | | | **2** | **33,3** |
| | | | **3** | **59,8** |
| **type 4** | Double leaf wall with façade in burnt clay hollow bricks mechanically fixed to a metallic structure | **60%** | **4** | **37,6** |
| **type 5,6** | Double leaf suspended façade made with terra cotta tiles | **75%** | **5** | **77,6** |
| | | | **6** | **76,1** |
| **type 7** | Double leaf suspended façade made with large ceramic tiles and linear joints | **75%** | **7** | **75,3** |
| **type 8** | Double leaf suspended façade made with large ceramic tiles and punctual joints | **77%** | **8** | **80,3** |

After this first evaluation we performed a more detailed analysis using Analytic Hierarchy Process[v]. This decision help tool provides a very flexible assessment framework, in which criteria are organised into a decision tree algorithm. This phase has been conduced first with a comparison among criteria, used to produce a weight (relative importance) for each of them, then with a comparison between each couple of solutions for each criterion. Comparative judgements have been given with both qualitative[vi] and numerical values.

The general assessment on durability that our research produced on the listed technical solutions for the building façade is synthesised in table 2. These results, in addiction with the quoted post occupancy evaluation, lets us suppose a minimum value for the life cycle of each system and a maximum target for designers in the correct construction, use and environment conditions.

These values show sometimes a reduction of the estimated life cycle respecting to previous researches[vii] and more in general they give a detailed evaluation of differences among similar solutions.

**Table 3. Analytic assessment of durability**

| | durability of building elements | technological and functional complexity of building systems | critical compatibility of materials | Post occupational evaluation | minimum LC years | Predicted LC years |
|---|---|---|---|---|---|---|
| Type 1 | Deterioration of insulating pannels: their performances can be halved in 20 years by an excess of water in the cavity | Cavity should be protected by rain water and damp must be dried out by flashing or ventilation | Chemical effect of salts on burnt clay bricks: this can produce fragility and breach in bricks | Deterioration of burnt clay bricks caused by evaporation of the rain and hygroscopical salts; | **30** | **80** |
| Type 2 | Excessive fragility of external layer: it can be braked by mechanical agents or by thermal variation | Tolerances among different layers of the external leave | Excess of differences in damp permeability among materials in the external leave | Breaches of elements caused by accidental hits or thermal variation or freezing; | **20** | **60** |
| Type 3 | Deterioration of insulating pannels: their performances can be halved in 20 years by excess of water in the cavity; deterioration of metallic ties | Number and flexibility of metal ties | Metal ties corrosion caused by chemical solutions of rainwater and burnt clay salts | Lesions produced by structural movements among façade and inner leave, produced by metal ties rigidity | **30** | **70** |
| Type 4 | Deterioration of metal substructure; mechanical deterioration of rubber rings among hollow bricks of the external pannels | Tolerances among external leave and metal frames; loads transmission among substructure and building structure | Deterioration of rubber rings among hollow bricks of the external pannels caused by UV rays; metal ties corrosion caused by chemical solutions of rainwater and burnt clay salts | Fractures and deformations produced by structural movements among façade and inner leave | **20** | **60** |
| Type 5 | Deterioration of metal substructure; Deterioration of terra cotta caused by water imbibition and drying | Tolerances among external leave and metal frames, loads transmission among substructure and building structure | Metal ties corrosion caused by chemical solutions of rainwater and burnt clay salts | Corrosion of galvanised steel substructure and other metallic accessories | **30** | **80** |
| Type 6 | Deterioration of metal substructure; Deterioration of terra cotta caused by water imbibition and drying | Tolerances among external leave and metal frames, loads transmission among substructure and building structure | Metal ties corrosion caused by chemical solutions of rainwater and burnt clay salts. | Corrosion of galvanised steel substructure and other metallic accessories; breaches of tiles caused by accidental hits in thin façades; | **20** | **80** |

| | | | | | | |
|---|---|---|---|---|---|---|
| Type 7 | Deterioration of metal substructure; | Tolerances among external leave and metal frames, loads transmission among substructure and building structure | Metal ties corrosion caused by chemical solutions of rain water and construction products | Corrosion of galvanised steel substructure and other metallic accessories; Facing pannels collapse caused by linear joints reactions; | **30** | **80** |
| Type 8 | Deterioration of metal substructure; | Tolerances among external leave and metal frames, loads transmission among substructure and building structure | Metal ties corrosion caused by chemical solutions of rain water and construction products | Corrosion of galvanised steel substructure and other metallic accessories; | **40** | **80** |

## 5 CONCLUSIONS

From this comparison among solutions of double leaf suspended façades it's possible to point out some interesting notices for designers and clients.

First of all double leaf façades made with an external leaf in mechanically assembled ceramic or terra cotta elements can assure good performances in comparison with traditional double leaf walls with cavity (see Table 2). Their durability is generally good (see Table 3), but doesn't exceed traditional double leaf wall one, and the service life can be improved by a re-use of building elements, made possible by their mechanical, *dry*, assembling.

Therefore It is clear that the first reason for which these systems are adopted is usually related to their architectonic appearance, because the high cost of their construction doesn't currently satisfy a value analysis (Maffei & Boccaccini, 2000(i)).

For what concerns durability we need to underline that some façade elements present an excessive fragility to accidental hits and deformations, not always compensated by the easiness in elements substitution. The adoption of large pannels and the use of ceramic instead of burnt clay can assure a longer life cycle with less risks of failure.

To obtain the planned sevice life it is necessary a careful design and forecast of loads interactions among substructure and closure systems: this can be very difficult when the façade is built on an existent construction, as typically happens in rehabilitation of residential buildings (Grassi 2000). Designers must check out that all the joints between façade and the inner leave are realised in stainless steel; the use of stainless steel must be diffused also for all kinds of elements used for the maintenance (ties, screws, etc.). Joints betweeen pannels and structure must be preferably punctual, made with rivets on the back of facing tiles: other joints, like linear ones, must be avoided.

On a wider point of view we can underline that double leaf suspended façades usually answer a very actual requirement, the reduction of the environmental impact in the service life, particularly because:

- these systems can be perfectly dismantled and all single elements can be recycled
- if ventilated, those systems can induce energy savings.

At last we should underline the good flexibility in use of these systems, very useful for their applications in the functional and architectonic improvement of existing buildings.

From a methodological point of view it is possible to perform an evaluation of the presumed durability of building elements already in the brief phase, but only in a multidisciplinary activity that brings around the decision table all the single experts of the design team. In this sense we encourage the development of software like the quoted DAEC, tools that can be very useful if applied in a multidisciplinary activity.

## 6 ACKNOWLEDGMENTS

We would thank Prof. D.A. Langford, Branka Dimitrijevic and all the research team of Strathclyde University for the diffusion of DAEC evaluation tool; we also thank Ing. Ennio Grassi, GS Enineering, for images in figure 1 and for his personal comments and suggestions.

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## 8 NOTES

[i] Building for Environmental and Economic Sustenibility (BEES) is a tool produced by the National Institute for Standards and Technology of U.S. Governement for the evaluation of sustenibility of buildings materials and components. The tool provides a decision support system based on the environmental assessment suggested in ISO 14040. Fourther informations on www.epa.gov.

[ii] One of the last accident has occurred at the *Arché de la Defense*, Paris. Last winter some stone tiles mechanically fixed with linear joints fell down because of a fragile collapse produced by wind cycles of pressure. The whole north façade should now be repalced, after 20 years of service. The lack of resistence to cyclical loads of linear joints (see fig. 1) is now proven and designers should avoid this kind of solutions in ventialted façades.

[iii] This research has been performed with group activities with the aim of completing the brief phase of new Pisa's District office building. Users needs, technical requirements and building performances have been defined by interdisciplinary workshops.

[iv] The main outcomes of the research project recently completed by D. Langford, I. MacLeod, T. Maver and B. Dimitrijevic and the assessment tool for durability, adaptability and energy conservation of buildings (DAEC tool) are available on a CD. For fouther informations please contact: branka.dimitrijevic@strath.ac.uk. or visit the site: http://www.ce.strath.ac.uk/sustain/

[v] We adopted a software implementation of AHP made by Gary L.Lilien and Arvind Rangaswamy and included in a set of tools for marketing engineering.

[vi] Qualitative; i.e. with respect to criteria 1 - adaptability - solution 2 is moderately better than solution 1; quantitative; i.e. with respect to criteria 4 - life cycle cost - solution 1 scores 1,35 over solution 2

[vii] The average life cycle for thin façades was extimed, for istance, in 48 years by a reseach of CSTC (CSTC, Recherches sur les performances du batiment, Compte rendù d'étude et de recherche n.29/1976).

# Durability Of Acrylic Waterproofing Membranes For Wet Areas

**BL Schafer**
**CSIRO Appraisals Victoria Australia**

**Summary: In recent years, environmental pressures have resulted in increasing use of acrylics as waterproofing membranes. A major barrier to the use of these materials has been concerns about the service life that they will achieve. CSIRO Appraisals has developed a testing protocol for the evaluation of the long-term serviceability of these materials, and this paper discusses the testing protocol and the interpretation of the results, including how they can be used to determine the suitability of a membrane to protect different substrates. As acrylic waterproofing membranes have a wide variation in their water vapour transmission rate, the paper includes the additional testing required to determine the suitability for use over substrates according to the water vapour transmission rate.**

**Membrane systems have a large variation in their elongation at break, thus requiring different types of bond breakers to cater for these varying elastic properties and thereby protect the membrane from fracturing at substrate discontinuities. The different types of bond breakers are discussed, along with their compatibility to the different elastic properties of the membrane systems.**

**The effect of curing time on acrylics is vital to the performance of the system. The lack of a sufficient curing time is one of the problems that has been highlighted in field monitoring that CSIRO Appraisals undertakes on membrane systems. The results from laboratory testing and those found in the field monitoring are discussed.**

**The test method developed by CSIRO Appraisals is currently being adopted by Standards Australia for inclusion in the Building Code of Australia as a deemed-to-satisfy requirement for the physical properties of membranes used in wet areas of buildings.**

**Keywords: Waterproofing membranes, wet areas, Building Code of Australia, bond breakers.**

## 1. INTRODUCTION

In recent years, environmental pressures have resulted in increasing use of acrylic-based materials for waterproofing membranes. A major barrier to the use of these membranes has been concerns about whether they will have an acceptable service life. CSIRO Appraisals has developed a testing protocol for the evaluation of the long-term serviceability of these materials when used for waterproofing wet areas. This paper disucsses the testing protocol and interpretation of the results, including how they can be used to determine the suitability of a membrane to protect different substrate materials.

The testing program has been developed as a result of CSIRO Appraisals needing to evaluate membranes for fitness for purpose, so the four membranes referenced throughout the paper are required to be kept confidential and will be referred to as A, B, C and D.

## 2. CHEMICAL RESISTANCE

The practice in Australia is to lay the waterproofing membrane under the tile bed. Thus, the membrane needs to function below a saturated tile bed. Solutions of chemicals used within the wet area can find their way to the surface of the membrane. A critical position where waterproofing membranes need to function is at the wall/floor junction, where the membranes need

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to cater for the movements that occur between the changes of substrates. Leakage at this location can result in water penetration into adjoining rooms, which means non-compliance with the Building Code of Australia. Even though the recommended tile-laying procedure is for the floor tiles to be laid under the wall tiles, in practice it is often the opposite, with the floor tiles abutting the wall tiles. While both methods will allow cleaning chemicals to track down to the membrane, this non-preferred tile-laying method makes the membrane more vulnerable to attack from cleaning chemicals. With the floor tiles abutting the wall tiles, cleaning chemicals that run down the wall tiles will enter the vertical crack that, with the passage of time, develops between the wall and floor tiles. Thus, it is imperative that the waterproofing membranes used in wet areas have resistance to the commonly used cleaning chemicals. The most commonly used mould growth chemical is sodium hypochlorite (bleach), while detergents are also used in the cleaning process and so resistance to detergent is also required. As the tile bed is fully saturated, long-term resistance to a head of water above the membrane is also critical.

The testing method developed uses deionised water for the water immersion testing. For all three testing exposures, 12 specimens are fully immersed in a sealed container with 600 ml of solution. Three specimens are tested at each of the periods of exposure, and there are three spare specimens in case of any breakages before testing. The specimens are tested in tension to the testing method given in Australian Standard AS 1145–2001 (Standards Australia 2001) using a type 5 specimen. The specimens need to include reinforcement where this is part of the membrane.

The testing method uses more concentrated solutions of the chemicals than would normally be applied when cleaning. For sodium hypochlorite a 2% solution is used – about three times more concentrated than is commonly used in mould-removal solutions. For the detergent, a 30% solution of an industrial-grade detergent is used. The use of more concentrated solutions is somewhat similar to the testing of materials used externally where, with accelerated weathering, they are exposed to high levels of UV in short-term testing to give long-term predictions of performance. However, for the chemical testing, it is expected that significant changes in properties will occur. What needs to be determined using this accelerated exposure is whether the membrane will lose its integrity to the point where it will fail to provide an acceptable service life.

What is an acceptable service life for these membranes? They are installed under an expensive tile or stone finish, and thus they need to have a service life equal to at least that of their expensive covering. No one really expects to replace the tiles or stone covering, that can cost more than 10 times that of the waterproofing, because of a membrane failure. While the tiles or stone covering could have a service life greater than 25 years, in reality by this time it is most likely that they would have been replaced to bring them into a more fashionable scheme. During a renovation when the covering is replaced, the membrane would be damaged, and would also need to be replaced. Bathrooms are one of the more personal areas of the dwelling and when there is a change of ownership, the bathroom is usually one of the first rooms to be renovated. Real estate figures give the average ownership of a dwelling in Australia's capital cities as less than 10 years. Thus, a 25-year service life for the membrane would easily meet a 95% confidence limit that it would be replaced within this time.

Initially, it was thought that testing at 7, 14 and 28 days would give all the data required. But as can be seen in Fig. 1, there is a reversal of the trend sometimes after 7 days and, to get a reliable result, the testing was extended to 56 days. In these tensile test figures, the ordinate is a comparison of the exposure result to that of the control for the sample under test. Using this ratio enables both elongation at break and tensile strength to be graphed on the one plot for comparison. In other figures, it enables direct comparisons of the relative performance of different membranes. In the initial development of the testing method, the 14-day testing increment was continued. However, as the method was to become a standard test, and costs needed to be contained, the test exposure times finally chosen were 7, 28 and 56 days. The 7-day exposure was used in preference to 14, as it will show early if a membrane has little chance of meeting the test requirements.

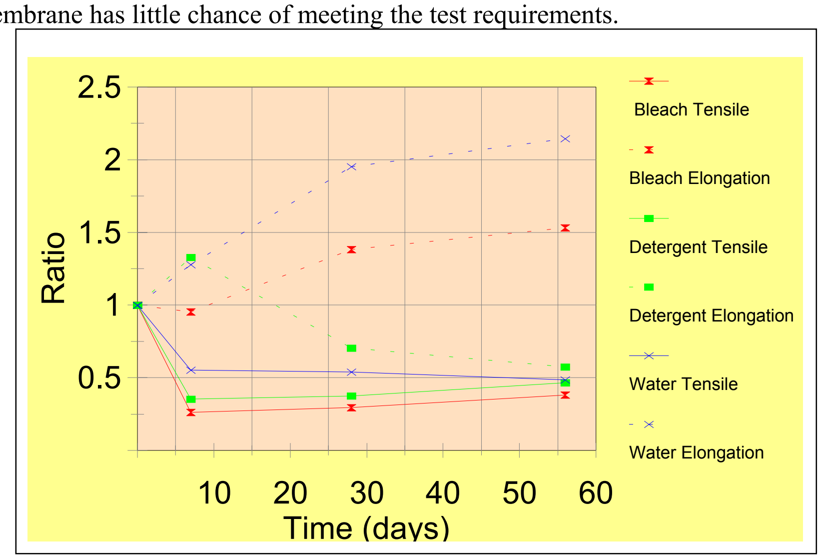


**Figure 1. Tensile strength and elongation at break results for membrane C**

For the membrane to remain serviceable, it is important that it maintains enough flexibility to enable it to accommodate any movements that occur over changes in substrates that it bridges. Thus, it is elongation at break that is the most important property to be maintained. Often, the membrane softens during exposure to both water and bleach, as happened with membrane C (Fig. 1). For both of these exposures, membrane C has had an increase in elongation at break with a corresponding reduction in the tensile strength. After the initial 7-day exposure to the detergent, this is not the case, with a loss of both elongation at break and tensile strength. A loss in both properties indicates that the membrane material is suffering some fundamental loss of its intrinsic property. In the case of membrane C, there is not sufficient loss under exposure to detergent to be of concern, with it keeping just over 50% of its initial elongation.

The results for membrane A are shown in Fig. 2. These are not nearly as convincing, with the continual loss of both properties on exposure to both bleach and detergent continuing significantly between the 28- and 56-day results.

The results for elongation at break for all four membranes exposed to detergent are given in Fig. 3. Membrane B shows a significant change in the results between 7 days and both 28 and 56 days. This reversal was observed for many of the membranes that we have tested and is an example of why extending the testing out to 56 days, rather than finishing at 28 days as originally proposed, is warranted. The results for tensile strength for all four membranes exposed to detergent are shown in Fig. 4. The rapid loss of both tensile strength and elongation at break for membrane B at 7 days, having lost over 50% of its elongation at break and about 50% of its tensile strength, would indicate that this membrane was unlikely to meet its long-term durability requirement. However, by 56 days it had virtually recovered all of it elongation at break while still retaining 25% of its tensile strength. Based on these longer exposure results, it was considered to be a satisfactory membrane for use in wet areas. The testing, as mentioned earlier, is using more concentrated solutions to show long-term effects to exposure to less concentrated solutions likely to be experienced in normal usage.

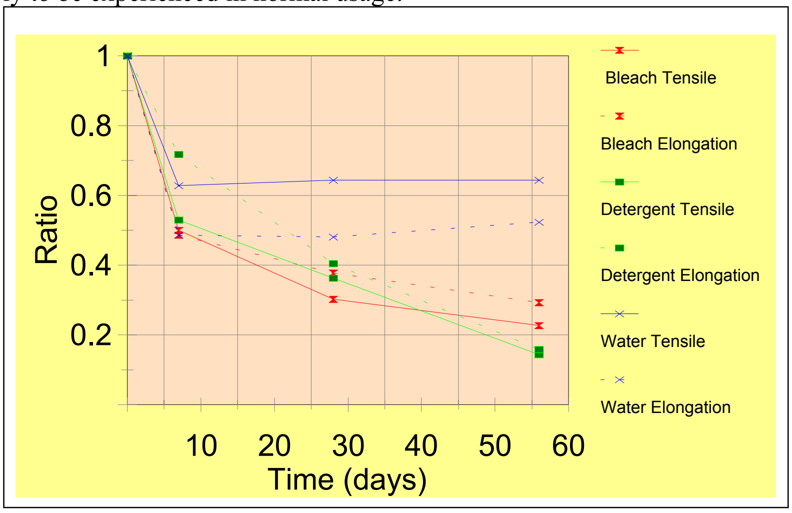


**Figure 2. Tensile strength and elongation at break results for membrane A**

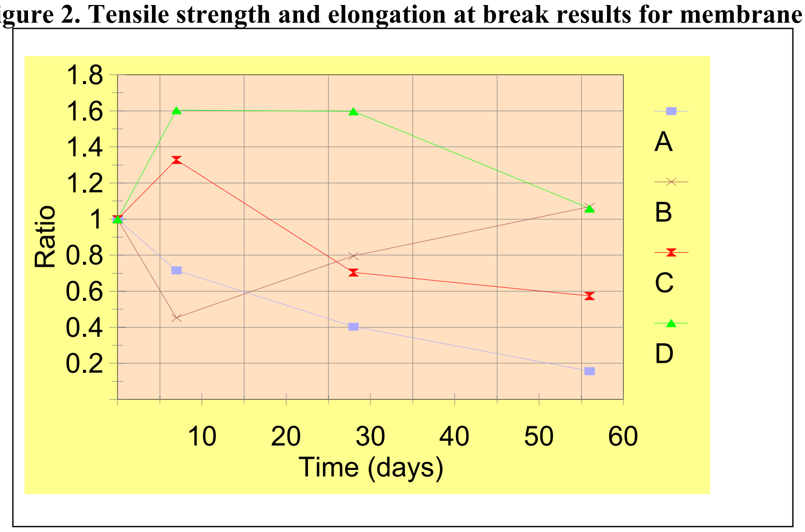


**Figure 3. Elongation at break for all four membranes on exposure to detergent**

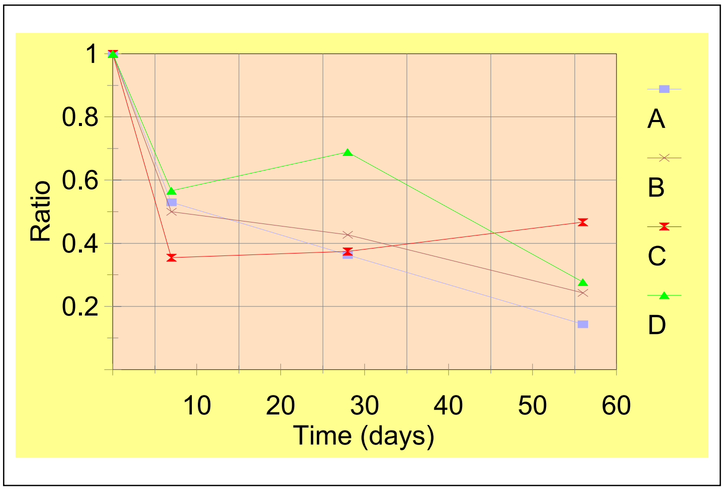


**Figure 4. Tensile strength for all four membranes after exposure to detergent**

The pass/fail criteria finally chosen was to require that at least 40% of the elongation at break needs to be retained after 56 days exposure. It was considered that if the specimen had retained sufficient strength to enable it to be tested, then it had retained sufficient tensile strength. In practice, with correct specification of bond relief over areas where the membrane is expected to be stretched, the membrane will only fail if it is stretched beyond its elastic limit, as given by its elongation at break property. The results for all four membranes for their elongation at break after exposure to bleach is given in Fig. 5, and after exposure to water in Fig. 6. Thus, membranes B and C passed the test criteria while A and D failed to do so.

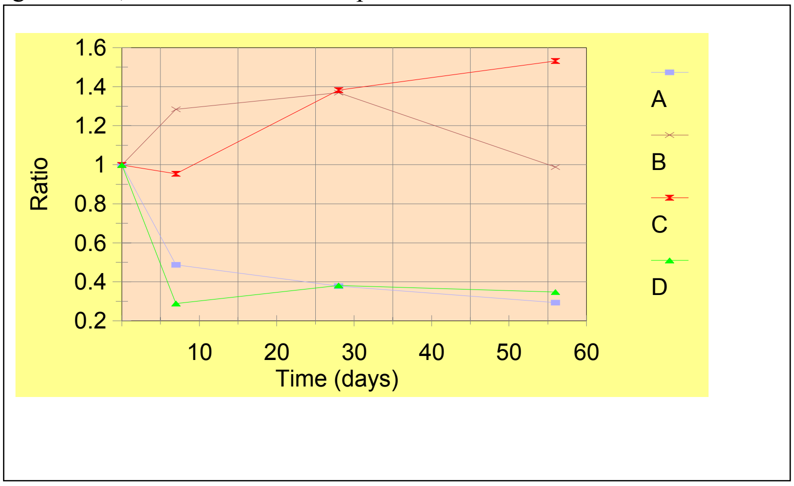


**Figure 5. Elongation at break for all four membranes after exposure to bleach**

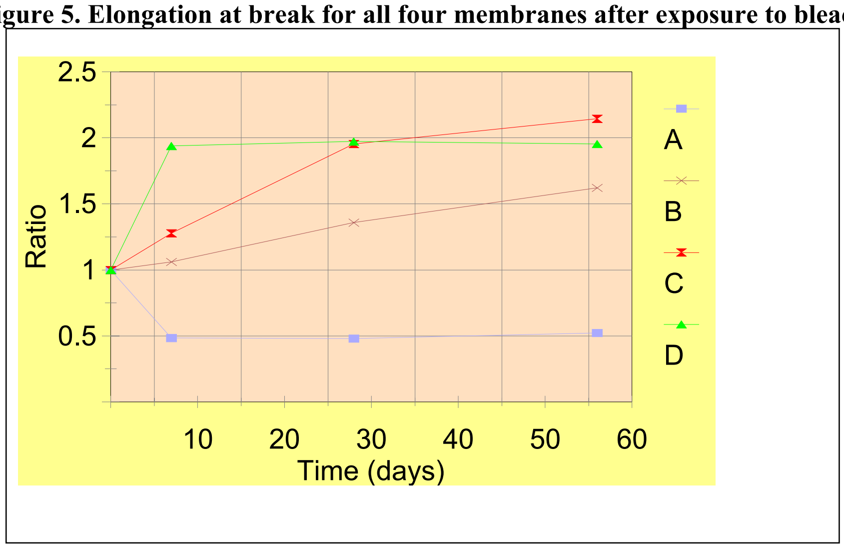


**Figure 6. Elongation at break for all four membranes after water exposure**

Some membranes that we have received for testing reach a stage of collapse at 7 days exposure and are not even able to be tested as shown in Fig. 7. The three on the left were exposed to bleach, while the one on the right was exposed to detergent. The worrying fact is that these were membranes that had been marketed for some time before they were tested for durability.

**Figure 7. Tensile test specimens that failed before they could be tested**

## 3. FATIGUE FRACTURES

The membrane will have to continue to bridge over any cracks and gaps between changes in substrates, while these gaps have potential for ongoing movements with changes in temperature and/or moisture content. Martin (1977) described a test for membranes that have high elongation at break. This method can be used for unreinforced acrylic membranes that have elongations at break well over 300%. However, when they have some reinforcement, or are a combination of cement and acrylic resin, their elongation at break can be well below the elongation required by the test as described by Martin. What we have done is to modify the Martin method to cycle the membrane over 75% of its elongation at break rather than the 200% as given in Martin's method. The specimen includes any reinforcement that is used. This modification has developed a satisfactory method to check that the membranes do not fatigue fracture where they bridge over moving gaps.

To make sure that the membrane is kept within the movement it can accommodate, there is a need to give bond relief from the substrate at points of discontinuity subject to movement. Three classifications of membranes have been defined.

Class 1 membranes are those that have an elongation at break of less than 65%. Due to their low extension before they fracture, these need a bond breaker that takes up the movement by bending the membrane rather than stretching it. The bond breaker should be a backing rod utilised to place an initial curve in the membrane. The rod allows the membrane to straighten when a gap widens or become more curved to tolerate a shortening of the gap. Figure 8 shows this type of bond breaker.

Class 2 membranes are those that have an elongation at break between 65% and 200%. These need a bond breaker with a suitable unbonded length to allow the membrane to stretch to accommodate the movement. The width of the tape is determined so that the membrane will not go beyond 65% of its elongation at break to take up the movement. Figure 9 shows this type of bond breaker.

Class 3 membranes are those that have an elongation at break greater than 200%. These hardly need a bond breaker at all, but any bond relief will lower the risk of failure. A fillet of sealant will give all the bond relief that is required. Figure 10 shows this type of bond breaker.

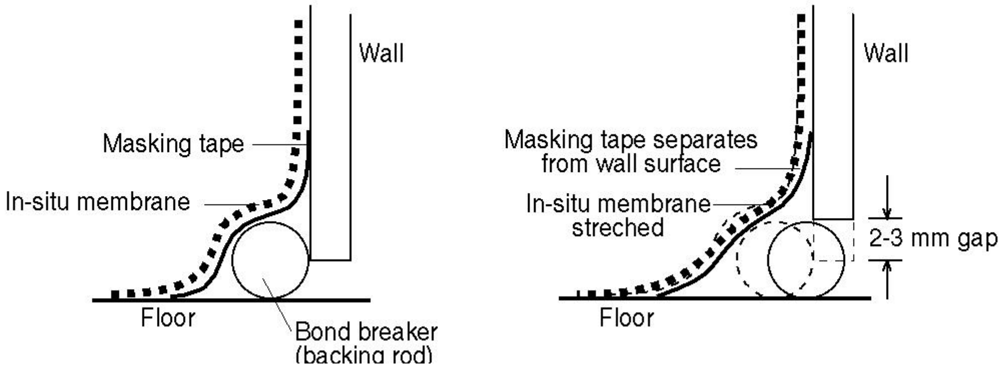


**Figure 8. Bond breaker for a Class 1 membrane**

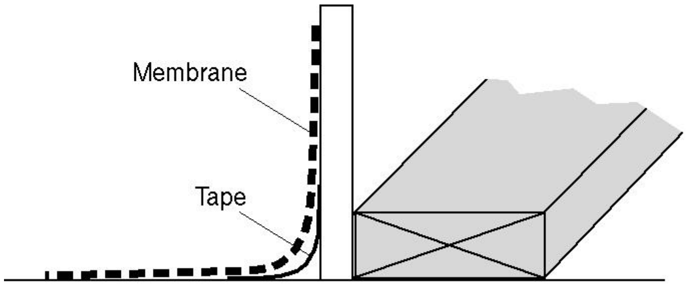


**Figure 9. Tape bond breaker for a Class 2 membrane**

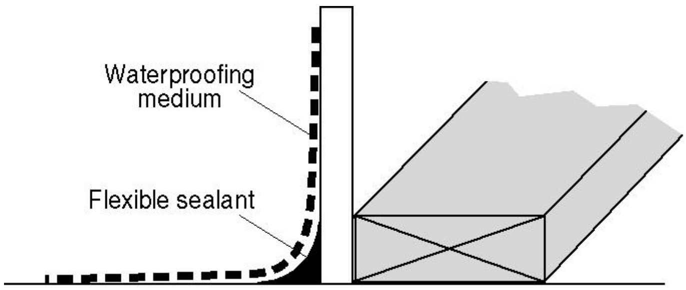


**Figure 10. Flexible sealant bond breaker for use with a Class 3 membrane**

## 4. SUBSTRATE PROTECTION

Acrylic membranes have high water vapour transmission rates. The actual rate for any membrane depends on its formulation and can be as low as 2 g/m²/24 hr  to in excess of 60 g/m²/24 hr. For those at the upper end there is potential for moisture to build up in the substrate under the membrane by water vapour transmission through the membrane. To ensure that membranes do not cause this problem to the substrate, especially those subject to moisture damage, a ponding test was designed. In the test, a 600 mm square tray of the membrane is formed over a particleboard floor substrate which has a butt joint running across the middle of the tray. Insulated probes are installed from the underside of the particleboard to measure the moisture content 1 mm under the membrane by an electrical moisture meter. The probes are located at set distances each side of the butt joint in the particleboard. After the tray has fully cured, it is flooded with water to a depth of 25 mm and the moisture content recorded until it has stabilised. For the first five days, daily readings are taken, and thereafter twice a week (3 and 4 days between readings). Simultaneous readings are also taken for a control piece of particleboard, which is placed beside the test tray. Both the test tray and the control are located in standard laboratory conditions of 23°C and 50% RH. Figure 11 shows some of the results to date recorded from this test. Clearly the 66 g/m²/24 hr material was showing a real problem, with an increase in moisture content in the particleboard of nearly 30%, while the 2 g/m²/24 hr material had little increase and was of no concern.

It is recommended that shower recess membranes not be used directly on top of particleboard where test results give a moisture content increase of 10% or greater than that of the control.

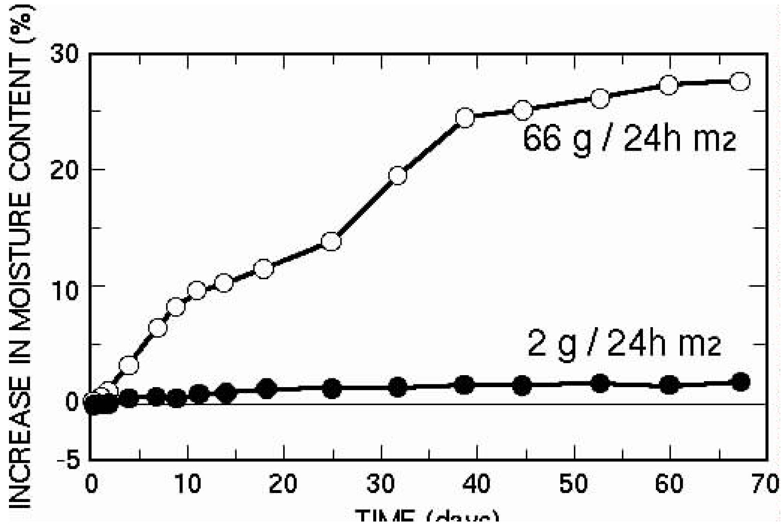


**Figure 11. Moisture increase in particleboard flooring under different membranes**

CSIRO Appraisals has collected enough data to know that if the water vapour transmission is 8 g/m²/24 hr or less, then the membrane can be used directly over particleboard under a shower. This limit may be raised in the future when more data is available for membranes above the current limit of 8 g/m²/24 hr.

## 5. CURE TIME

One of the main causes of failures of acrylic membranes that CSIRO Appraisals has found in field inspections is inadequate curing before the membranes are covered. Acrylics rely on water loss to cure, and when used in wet areas, such as small and poorly vented rooms (e.g. en suites), it can take over 7 days for a straight acrylic to cure in winter conditions. If covered before they are allowed to cure fully, they can re-emulsify where they are continually wet, as is the case when used in shower recesses. To retain the elastic properties, one either needs to wait for a full acrylic to cure or use heating and airflow to speed up curing. We stress in all our appraisal reports that slow curing has to be allowed for, but still we occasionally find installations where they have been covered without being cured.

To shorten the cure time, there are some two-part formulations consisting of a liquid acrylic mixed together with a cement-based powder. These will cure within 24 hours, but they loose the high elongation that is available with a full acrylic formulation.

One of the main problems with the cement–acrylic formulations is that, due to their thick creamy nature, they can be applied too thick, which results in drying crazing. This is caused by the skin curing before the body of the membrane cures.

Straight acrylics used without reinforcement can be applied too thinly. Some inspections have found a thickness of less than 0.1 mm where an unreinforced acrylic has been used. The usual specified thickness is about 1 mm.

## 6. CONCLUSION

CSIRO Appraisals has developed a testing program that has a proven record in being able to successfully evaluate the suitability of acrylic membranes for use in the waterproofing of wet areas within buildings. The testing program is currently being adopted into an Australian Standard. Several membranes have been found to be unsuitable for use in wet areas. We are now in a position to make sure any membrane that is proposed will give an acceptable service life.

## 7. References

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# Performance Evaluation Of Photocatalyst Coating Systems Under Outdoor Exposure Condition

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**Summary: Application of "Photocatalyst" (typical substance in this paper: titanium dioxide, $Ti0_2$) into the building materials and/or components, likely internal and external finishing boards, paint coatings, membrane materials, ceramic, film etc., is rather newly developed technology. One of the advantageous features by applying this photocatalyst would be the ability of self-cleaning or self-desoiling on the surface of materials.**

**This paper introduces the results of two series of experiments. In the first series of experiments, the effect of photocatalyst was confirmed through the decomposition of methylene blue in the laboratory by using ceramic tiles treated with titanium oxide. The effect was also confirmed after some degradation treatments for the tiles such as immersion into acid solution, immersion into alkaline solution, and sunshine carbon irradiation. In the next series of experiments, the outdoor exposure test under the pastoral and polluted ambient areas were conducted for the various exterior finishing materials treated with titanium dioxide.**

**Although relatively limited amount of data are available at present stage, the following conclusion could be made;**

**1) The effect of photocatalyst could be clearly recognized through fading out of methylene blue in the laboratory test even after various degradation treatments.**

**2) The self-cleaning effect was also recognized in the outdoor exposure test. The degree of effect was dependent upon each commercial product at present stage; however, products treated with photocatalyst showed the significantly higher self-cleaning effect compared with ordinal non-treated products.**

**3) In the evaluation of striped soiling which were often observed on the exterior envelope, the parameters derived from optical measurement of striped soiling possessed good correlation with the visual inspection results by using designated pictorial sheets. However, the correlation was becoming low in case that overall soiling was predominant on the specimens.**

**Keywords: Durability, Outdoor exposure test, Photocatalyst, Soiling, Titanium dioxide**

## 1 INTRODUCTION

Surface appearance of a whole building or a part of it should be one of the aesthetic requirement for the building as defined in ISO standard (ISO 6241:1984) as one of the "visual user requirement" which composed of several sub-factors as such colour, texture, regularity, flatness etc, and it has been one of the long-time major issue how to sustain the surface appearance in an acceptable condition, even though the acceptable level depend fully upon each building.

Nireki (2000) reported based on his experimental works, the soiling issue for membrane material leads to reduction of its light transmission coefficient and this directly effects the primary performance requirement to sustain the inner visual environment of large scale membrane structure.

As to the soiling of external envelope, it can be divided into the uniform and local one, that stand for uniformly soiled and partial or dotted soiling within a certain area of a plane of building or a component, here, in general the later one is much noticeable than the former one. For example, Motohashi & Sakai (1993) pointed out the soiling behaviour on the fluoropolymer paints as one of the important deterioration phenomena.

Much attention has been paid to cope with this contamination in building. Various countermeasures to protect soiling had been tried both from design aspect such as shape, dimension, details of external component and from material modification aspect by changing water repellent property, hydrophilic property, electrostatic property, resolvable property etc.

Application of the “Photocatalyst” would be one of typical examples. Titanium dioxide ($TiO_2$) type photocatalyst that aiming to apply to buildings, civil engineering and also medical engineering area, has been intensively developed.

The principal character of photocatalyst is capability of disintegrating the soiling substances which led to self-cleaning effect, and controlling fungi and bacteria, whereas, its applicable fieds has been expanding as such coating system for external wall, kitchen wall, bathroom, measures for surface contamination of lighting appliances and sundry goods, preventive measures for indoor air contamination especially due to VOC (volatile organic component), medical facilities, deodorant etc. according to Fujishima et al. (1999)

## 2 PURPOSE

The objective of this experimental research is to confirm the self-cleaning effect of exterior finishing materials treated with photocatalyst. The selected finishing materials were both commercial and under developing ones including ordinanary finishing materials as the references.

Rather large amount of photocatalyst applied materials are actually used in buildings at present. However, the effectiveness of these finishing materials has not been widely confirmed yet. In other words, although various photocatalyst applied products are put on the market, very little reliable technical information is available except for limited technical data from each manufacturer. In this context, this research was started in order to obtain the results of outdoor exposure tests that could be significant among building engineers and material manufacturers as well as customers.

The aim could be divided into the following three items;

- Performance evaluation of exterior finishing systems under in-use environmental condition to provide as the durability data that could be applicable for “service life prediction” in the principle of “service life planning” as specified in relevant ISO standard (ISO 15686-1) or AIJ guide (AIJ, 1989).
- Accumulation of some experimental data for setting of “Evaluation method for self-cleaning effect.”
- Accumulation of some experimental data for developing “the laboratory test” for evaluation of photocatalyst applied products.

## 3 OUTLOOK OF $TIO_2$ PHOTOCATALYST

As for the mechanism for the case of surface finishing materials, the interface of soiling contaminants adhered on the finishing materials could be decomposed by the effect of radical reaction led by the photocatalyst ( typical substance in this paper: titanium dioxide, $Ti0_2$ ) reaction that is accelerated under solar irradiation.

In practice this reaction would be active not only on the side of the irradiated surface but also on the opposite surface, therefore, it usually does need to apply a protective thin coating likely primer coating to protect the substrate if the substrate consists of organic substance. Consequently, when the photocatalyst treatment is applied to the organic material substrate, one layer of protective treatment against radical reaction is recommend before treatment with photocatalyst in the applying specification.

## 4 SPECIMENS

Type, dimension and some detail of specimens are shown in Table 1, Figure 1 and Figure 2.

**Table 1. Specimens**

| | Substrate | Finishing | Variation |
|---|---|---|---|
| A | Aluminum Sheet (1mm)<br>Acrylic-silicone coat (50μ) or<br>Polyurethane coat (60μ) | $TiO_2$ clear coat, 10-40g/m$^2$ | 8 |
| B | Aluminum Sheet (1mm) | $TiO_2$ paint coat (white), 15-30μ(60-100g/m$^2$) | 5 |
| C | PVC-coated membrane material | $TiO_2$ clear coat, 1μ(100g/m$^2$) | 1 |
| D | Aluminum Sheet (1mm) | $TiO_2$ masonry coat (white), 3-7Kg/m$^2$ | 1 |
| E | Aluminum Sheet (1mm)<br>Acrylic-silicone (50μ) or<br>Polyurethane coat (60μ)<br>(* as reference of A) | Ordinary Silicate clear coating (30g/m$^2$) | 6 |
| F | PVC-coated membrane<br>Material (*as reference of C) | PVF clear film (38μ) & none | 2 |
| G | Aluminum Sheet (1mm)<br>(*as reference of D) | Ordinary masonry coat (250g/m$^2$)<br>Aclylicsilicone (120g/m$^2$) | 1 |
| P | Ceramic tile | Titanium dioxide clear coat (20-30g/m$^2$) | 5 |
| Q | Ceramic tile (*as reference of P) | Hydrophile silicate clear coat (10-20g/m$^2$) | 4 |
| R | Ceramic tile (*as reference of P & Q) | None | 9 |

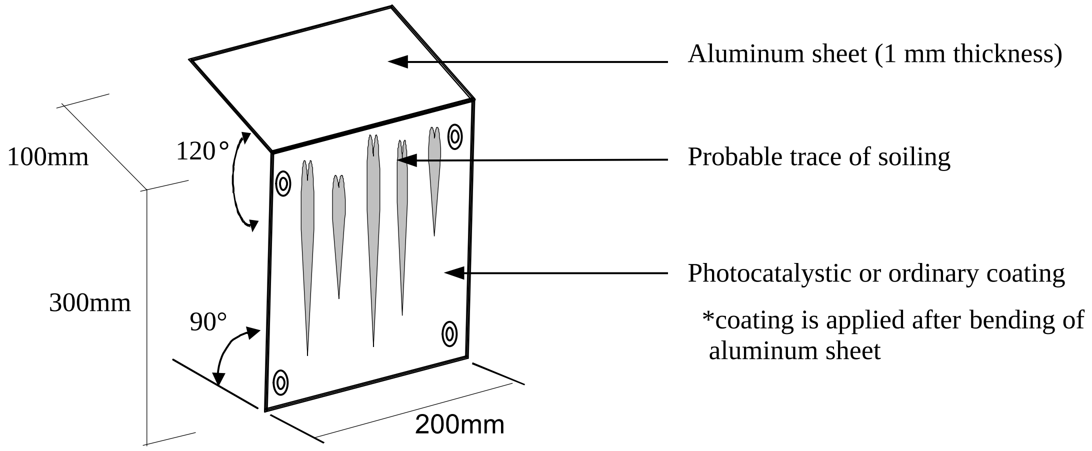



**Figure 1. Detail of specimen A to G**

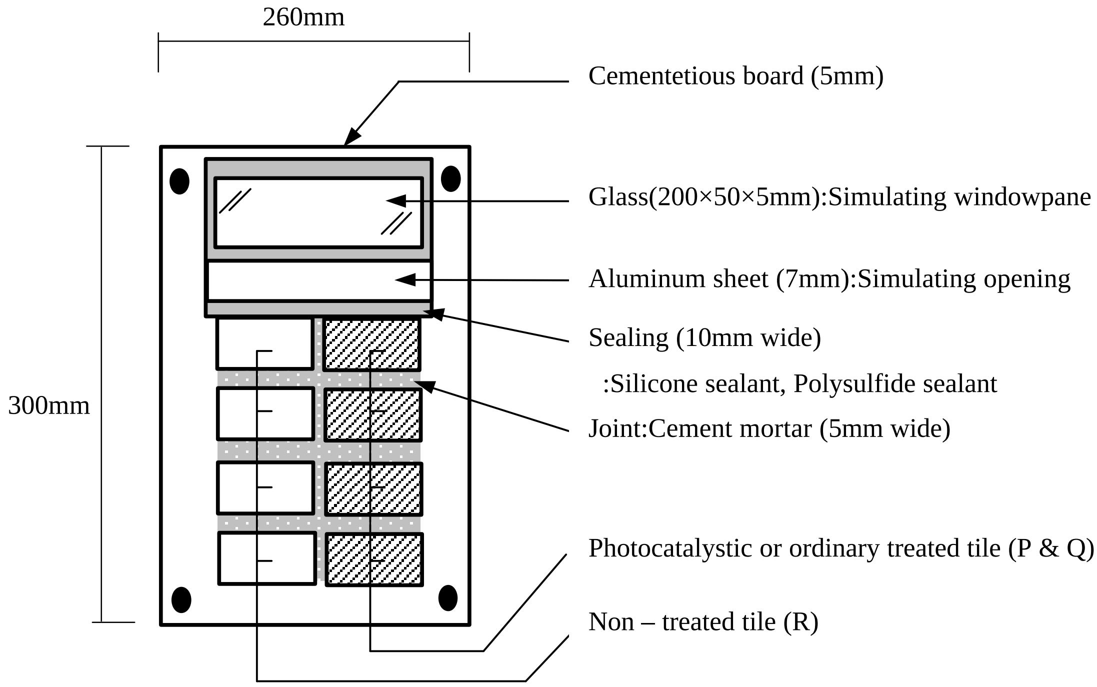



**Figure 2. Detail of specimen P, Q & R**

## 5 LABORATRY TEST FOR CONFIRMING DECOMPOSITUION OF METHYLEN BLUE

By using one of the specimens P in Table 1, activity of photocatalyst was confirmed through the decomposition of methylene blue stain which was coated on the one of the specimens P. This preliminary experiment was conducted before conducting the outdoor exposure test. The one of the specimens P in Table 1 (Porcelain tile coated with titanium oxide) were immersed in methylene blue ethyl alcohol solution (0.1 wt%). After raising the specimens from the solution, the specimens were dried for 30 minutes at 20? and 65%RH. Before immersion into the solution, some of the specimens were also underwent the following degradation treatment.

Immersion in acid solution : Immersed the specimens in 5 wt% sulfuric acid for 96 hours and washing with water.

**Immersion in alkaline solution : Immersed the specimen in saturated calcium hydroxide solution for 96 hours and washing with water.**

Sunshine carbon arc irradiation : Sunshine carbon arc irradiation in accordance with ISO 4892-4:1994 for 2000 hours.

After drying for 30 minutes, the specimens were irradiated by use of ultraviolet lamp (Toshiba FL40S-BLB). The ultraviolet irradiation intensity was 0.4mW/cm$^2$ at the surface of the specimens. Degree of fading of methylene blue on the specimens was monitored by measuring color difference (? E) compared with the initial values for the specimens before immersion into the methylene blue solution. The number of repetition for the specimens was three. The results were shown by use of the average value of the three specimens.

## 6 OUTDOOR EXPOSURE TEST

### 6.1 Exposure conditions

All types of specimens listed in Table 1 were exposed to following three different environments.

BRI : Outdoor exposure site, Building Research Institute, Tsukuba City (some 50 km north of Tokyo).

Kashiwa-1: Just around the tollgate facility of Kashiwa junction, Jyoban motorway (some 30 km north of Tokyo), and heavy polluted environment due to traffic.

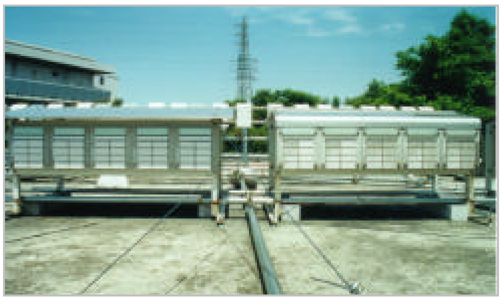
Kashiwa-2: On the roof of office of motorway in the vicinity of Kashiwa-1 slightly less polluted than Kashiwa-1.

In BRI and Kashiwa-2, all porcelain tile specimens were exposed both facing south and north in due consideration of different ultraviolet irradiation and specimens were attached on the rack installed around the gate hatch, faced to the parallel of traffic flow. (Photographs 1 to 6)

**Photograph 1. Overview of outdoor exposure for specimen A to G in Kashiwa-1 site**

**Photograph 2. Outlook of specimens A to G in Kashiwa-1 site**

**Photograph 3. Overview of outdoor exposure test in Kashiwa-2 site**



**Photograph 4. Outlook of specimen P, Q & R (tile) in Kashiwa-2 site**

*to the left:* with eaves, to the right: without eaves

**Photograph 5. Closing up of soiling behaviour,**

*the left row* in each block is non-treated tiles *and the right row* ; photocatalyst coating applied ones

**Photograph 6. Closing up of rain trace,**

*to the left:* specimen, *to the right:* pictorial designation sheet

### 6.2 Visual evaluation

Visual evaluation was conducted by using the pictorial designation system like Nireki (1986) had already applied for deterioration state of external masonry coating system. In this project, the pictorial designation of state of soiling for surface coating systems has been provided with intention to include evaluation of striped soiling mainly as a result of rain water flowing on the surface coating.

Striped soiling state is classified into five degrees from none to excessive on the basis of previous on-site investigation data on the soiling of surface finishing systems. Each degree can be divided into five sub-degrees in the order of soiling state within each degree and this can be printed on a sheet as in Figure 3, then, actual evaluation of soiling can be carried out by applying these five designation sheets.

The actual evaluation work was operated by 4 to 5 members with taking the pictorial designation sheets at around two meter apart from each specimen.

### 6.3 Measured Value [L*]

Evaluation in this project was focused on behaviour of soiling over time by the visual evaluation and measured value [L*]. Soiled state can be defined as a difference of value between the initial value and a measured value at certain time, here, the value [L*] is specified as a quantitative value in this project.

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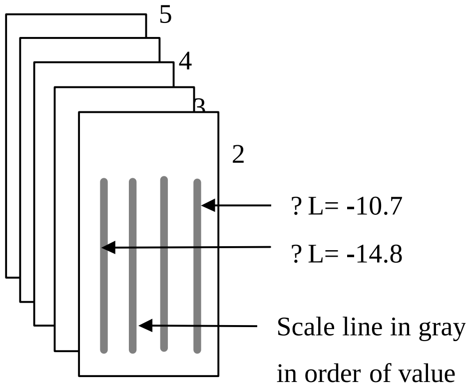


| Degree | [? L*] value |
|---|---|
| Degree 1 | -14.8 ~ -10.7 |
| Degree 2 | -5.6 ~ -3.1 |
| Degree 3 | -2.5 ~ -1.8 |
| Degree 4 | -0.8 ~ -0.3 |
| Degree 5 | None |

† [? L*]: Value measured by MINOLTA CR300

†† White base sheet [L*] = 92.0

**Figure 3. Pictorial designation of soiling states of surface coating system for visual evaluation**

The value [L*] was measured by using two different colour meters as MINOLTA CR300 (10.0mmf scope) and MINOLTA CR321 (3.0mmf scope), the former one is for relatively uniformly soiled area (Part with & without rain trace in Table 2) and smaller spotted area for the later (Part with rain trace only in Table 2). In principle, the value [L*] is an average out of three points within a plane of specimen. The soiling based on measured value [L*] in each part of specimen was defined according to Table 2

**Table 2. Soiling in each part of specimen**

| Part of specimen | Soiling |
|---|---|
| Part with & without rain trace | Difference between [L*(With & without rain trace)] with & without trace in an average of each area vs. initial value |
| Part with rain trace only | Value [L* (rain trace only)] in an average of the traced area vs. |

## 7 RESULTS AND DISCUSSION

### 7.1 Confirmation of photocatalyst in the laboratory test

The change in color difference in the laboratory test was shown in Figure 4 to Figure 7. Figure 4 shows the changes in color difference for the photocatalyst treated ceramic tiles and for original ceramic tiles. As shown in Figure 4, the color difference of the photocatalyst treated tiles was going down as irradiation time passed, which means that methylene blue was decomposed by activity of photocatalyst. On the other hand, color difference of the non-treated tiles was not going down, which indicates that the color of the specimens did not change. In other words, methylene blue was not decomposed on the non-treated specimens. Strictly speaking, the color difference of the non-treated specimens was going down very slightly; however, it is considered to be due to irradiation by ultraviolet lamp itself.

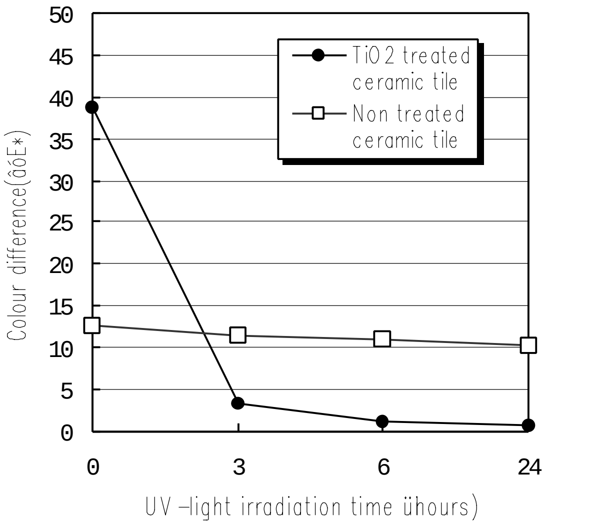


**Figure 4. Changes in color difference for the photocatalyst treated ceramic tiles and non-treated ceramic tiles.**

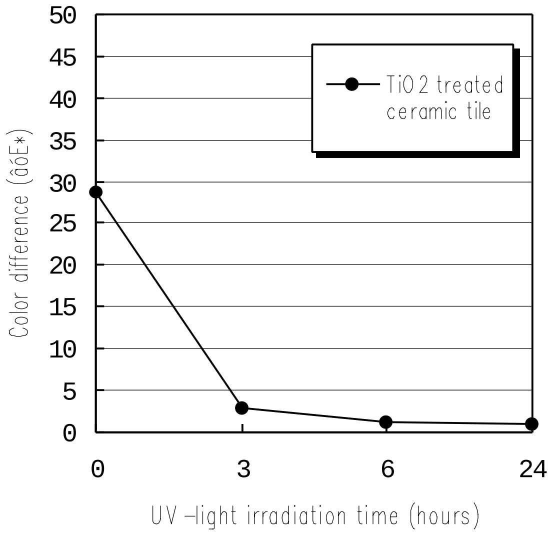


**Figure 5. Changes in color difference for the photocatalyst treated tiles after immersion in acid solution.**

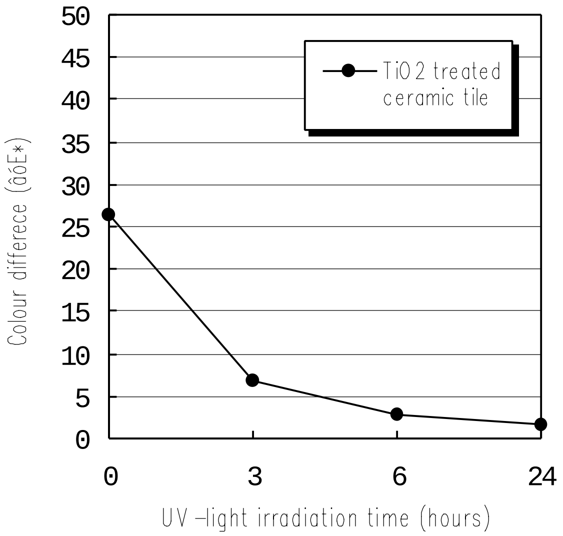


**Figure 6. Changes in color difference for the photocatalyst treated ceramic tiles after immersion in alkaline solution.**

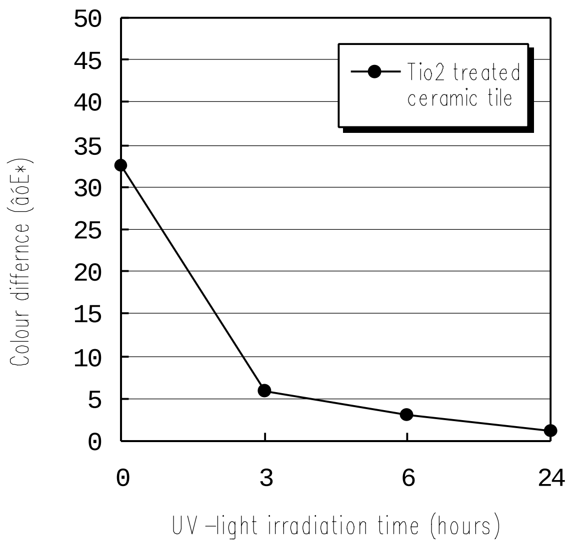


**Figure 7. Changes in color difference for the photocatalyst treated ceramic tile after sunshine carbon arc irradiation for 2000 hours.**

The decrease of color difference could be also confirmed in the specimens which were variously treated before immersion in methylene blue solution as shown in Figure 5 to Figure 7. It means that the activity of photocatalyst is still effective after these treatments.

### 7.2 Difference due to outdoor exposure environment

Difference of soiling behaviour among three environments can be clearly noticeable. Fig. 8 to Fig. 10 shows the percentages of the specimens at each degree. As shown in the Figures, it is evident that the exposure site is greatly affected the degree of soiling of the specimens. In the study, the most severe site is Kashiwa-1 where is near the tollgate of the high way. The next is Kashiwa-2 where is office in the vicinity of Kashiwa-1. BRI was the mildest site among the three.

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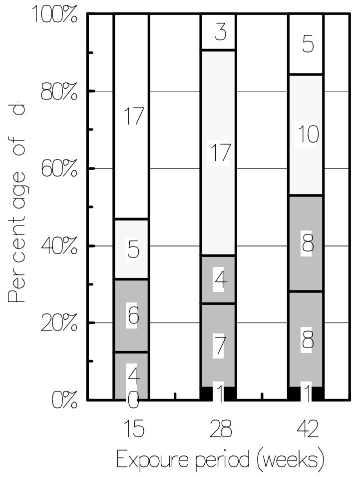


Figure 8. Result of visual evaluation at BRI, after 42 weeks (A to G)

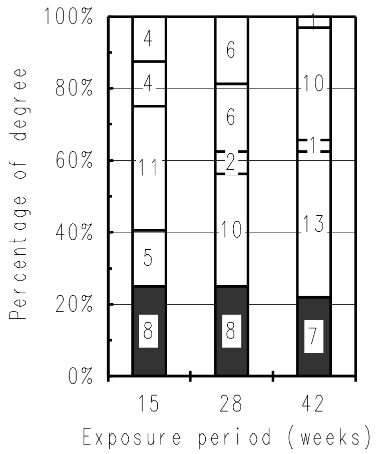


Figure 9. Result of visual evaluation at Kashiwa-1 site, after 42 weeks (A to G)

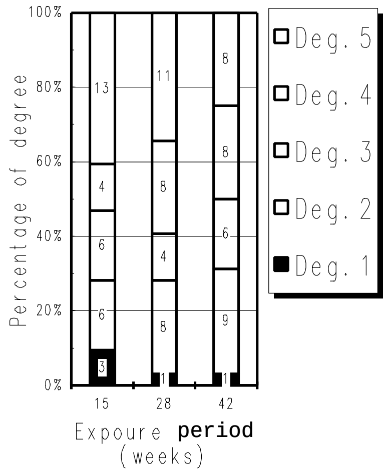


Figure10. Result of visual evaluation at Kashiwa-2 site, after 42 weeks (A to G)

8 specimens have reached to Degree 1 after 15 weeks in Kashiwa-1. In Kashiwa-2, 3 specimens have reached to Degree 1 after 15 weeks. On the other hand soiling has not yet reached to Degree 1 after 15 weeks in BRI. It can be said that degree of soiling is in the order of Kashiwa-1, Kashiwa-2 and BRI.

As to the difference due to exposure direction, north and south, specimen A to G, except G, show no distinctive difference by 42 weeks. This can be estimated that the sufficient photocatalystic reaction could be expected even in the north side where comparably lower solar irradiated environment than south. Nireki (1983) reported that the amount of total solar irradiation at surface of specimen facing north is around 40% that of facing to south in BRI outdoor exposure site.

### 7.3 Correlation between visual and [L*] value evaluation in outdoor exposure test

Taking the result as shown in Figure11 as an example, the correlation between visual and [L*(rain trace only)] value evaluation shows higher relation at 24 weeks exposure, however, as such high correlation can not be expected in due course of exposure days as shown in Figure 12. The main reason of this decreasing the coefficient of correlation should be dependent on the change of soiling mode on the surface of coatings, namely, noticeable rain trace in rather early time would no more be recognized due to the successive increase of uniform soiling over all area of surface over exposure term.

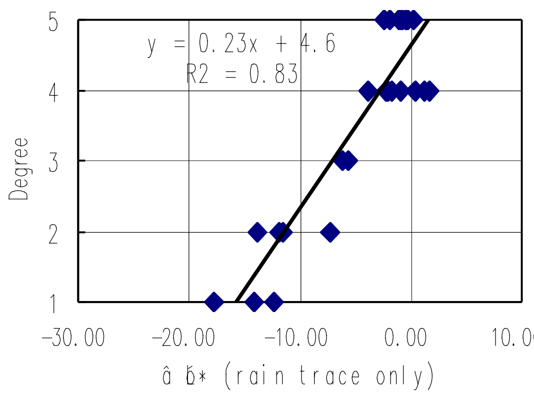


**Figure 11. Correlation between visual evaluation and value [L*],** measured in “Part with rain trace only” as in Table 2 (specimen A to G), after 24 weeks at Kashiwa-2 site

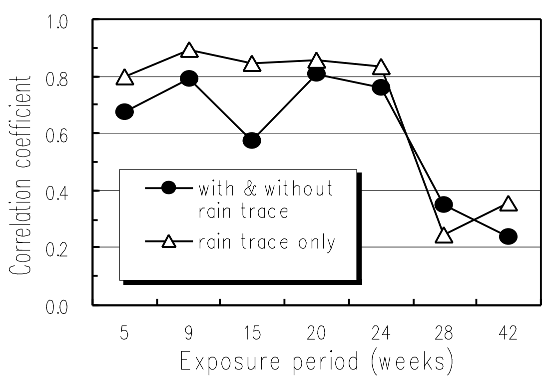


**Figure 12. Change of Correlation between visual evaluation and value [L*],** Specimen A to G, at Kashiwa-2 site, each symbol correlates to the order of soiling in Table 2

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As it is not shown in the paper, the same tendency was recognized for the [L*(with and without rain trace)] value evaluation. As far as the rain trace of the specimens are clear, visual evaluation by use of designated pictorial sheet, measurement of [L*(rain trace only)], and measurement of [L*(with and without rain trace)] shows good correlation; however, these values does not show good correlation in case that the uniform soiling is predominant on the surface of the specimens.

### 7.4 Effectiveness of photocatalyst

Taking result on the effect of photocatalyst as shown in Figure 13 (specimen A to G) as an example, effectiveness could be remarkable recognized throughout all photocatalyst applied finishing. It is also remarkable that the variation within the photocatalystic coating group can be noticed. The reason of radiation would be due to the quality of material itself, in fact, the quality of materials whether they are already developed or under developing stage. Take this rather special case apart, the fact, more than half of photocatalystic coatings have still keep at the level of degree 5 or at most 4, would shows the effectiveness of photocatalyst applied finishing in comparison with the results from another groups. As to the coating for tile (specimen P to R), the effectiveness can be recognized as in Figures 14 to 16.

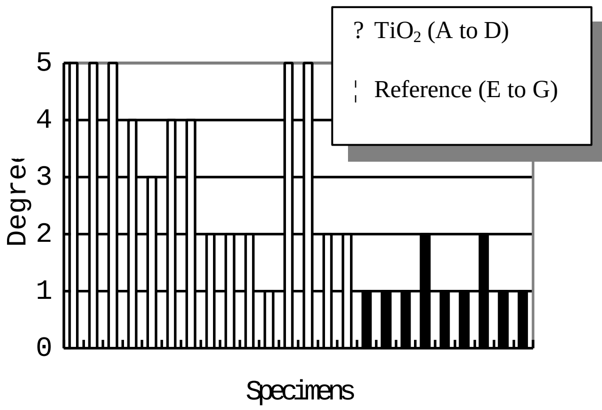


**Figure 13. Result of Visual Evaluation (specimen A to G), - after 19 weeks exposure at Kashiwa-1 site**

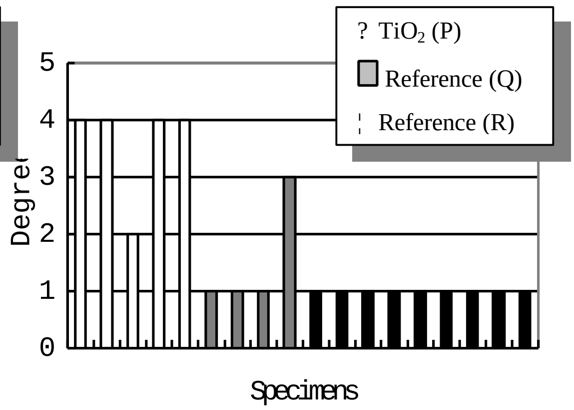


**Figure 14. Result of Visual Evaluation (specimen P to R), - after 42 weeks exposure at BRI**

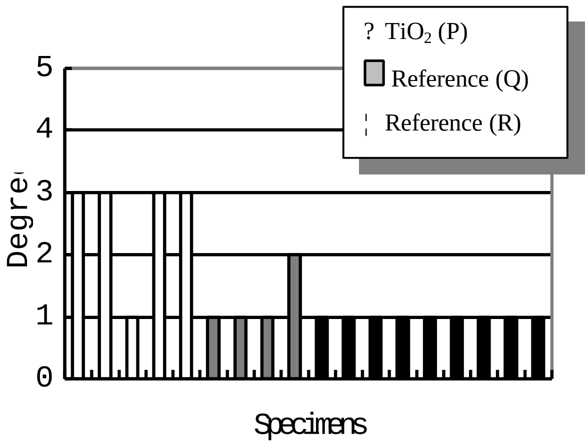


**Figure 15. Result of Visual Evaluation (specimen P to R), - after 5 weeks exposure at Kashiwa-1 site**

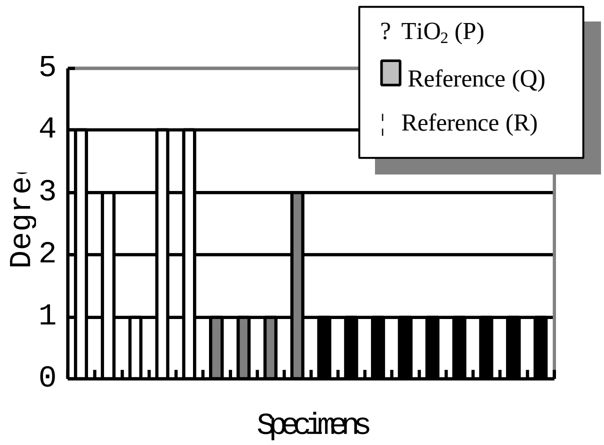


**Figure 16. Result of Visual Evaluation (specimen P to R), - after 42 weeks exposure at Kashiwa-2 site**

## 8 CONCLUSION

Application of photocatalyst into the building materials is rather newly developed technology. One of the advantageous features by applying this photocatalyst would be the ability of self-cleaning or self-desoiling on the surface of materials.

The purpose of this research is to confirm the self-cleaning effect of exterior finishing materials treated with photocatalyst. Nowadays, rather large amount of photocatalyst applied materials are actually used in buildings at present. However, the effectiveness of these finishing materials has not been widely confirmed yet. In other words, although various photocatalyst applied products are put on the market, very little reliable technical information is available except for limited technical data from each manufacturer.

Although relatively limited amount of data are available at present stage, the following conclusion could be made;

1) The effect of photocatalyst could be clearly recognized in the laboratory test even after various degradation treatments.

2) The self-cleaning effect was also recognized in the outdoor exposure test. The degree of effect was dependent upon each commercial product at present stage; however, products treated with photocatalyst showed the significantly higher self-cleaning effect compared with ordinal non-treated products.

3) In the evaluation of striped soiling due to rain trace which were often observed on the exterior envelope, the [L*(rain trace only)] and [L*(with and without rain trace)] shows good correlation with the result of visual evaluation by using the designated pictorial sheets. However, the correlation was becoming low in case that overall soiling was predominant on the specimens.

## 9 ACKNOWLEDGMENTS

A part of this paper refers to the research activities in “High performance external finishings” Committee (Chairperson, Dr. K.Motohashi) organized in the Japan Association of Building Research Promotion. Authors are grateful to Prof. Dr. T. Konish and Prof. Dr. Y. Masuda, University of Utsunomiya, for his continuous interest and helpful suggestions during implementation of this project, and we also acknowledge Dr. T. Nireki, Director, Tsukuba Building Test Laboratory, Centre for Better Living, for his contribution to the technical and analytical assistance for this project.

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# Rehabilitation For ASR Affected Reinforced Concrete Piers


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**Summary: Reinforced concrete columns were fabricated using the non-reactive river sand and reactive crushed stone. After the steam curing of 1 month, the columns were exposed outdoors for 1 year. When the width of cracks along the longitudinal steel reinforcement of the column due to the alkali-silica reaction (ASR) had attained about 0.5mm, three types of rehabilitation methods, surface coating, steel plate bonding and strengthening with prestressing wire, were applied to the ASR affected reinforced concrete columns. The changes in the expansion of concrete and steel reinforcement on the surface and in the center of the column were monitored for 4 years in order to investigate whether or not the rehabilitation methods applied can really control the expansion of concrete by confining the extension of cracks.**

**From the results of the experiment, it was clarified that the effect of surface coating in controlling the expansion of concrete was not significant compared with the controlling one without the surface coating, and that both the strengthening methods with steel plate and prestressing wire were effective in confining the extension of cracks and in improving the durability of concrete column.**



**Keywords: Alkali-silica reaction, rehabilitation, surface coating, steel plate bonding, PC confined method**


## 1 INTRODUCTION

In Japan, a lot of concrete structures, which were constructed in the 1960s and 1970s, are at present suffering from the serious problems associated with the durability of concrete, which are mainly the chloride induced steel corrosion and the alkali-silica reaction. ASR affected concrete structures were mainly repaired by the waterproofing agents on the surface of structures, but it was not effective from the results of the follow-up inspection after repair. In January 2001, the concrete committee of Japan Society of Civil Engineers (JSCE) published a new Standard Specification for Maintenance of Concrete Structures to deal with the rehabilitation and maintenance problem for concrete structures (Okada et al.1996, Concrete committee of JSCE.2001). On the other hand, it is also a matter of great concern to increase the seismic response ability of concrete structures after the Hanshin-Awaji earthquake disaster in January 1995. Concerning the strengthening method for concrete piers in bridge, the steel plate bonding method or carbon fiber sheet bonding method has been actively adopted in Japan. However, these strengthening methods are not always successfully adopted especially in ASR affected concrete piers because they usually have a high residual expansion ability. For this reason, a new-type strengthening method with prestressing steel wire, which was named the PC confined method, has been developed in 1997, and actually adopted in the strengthening for ASR affected concrete piers (Torii et al.1998, 2000). However, there are a relatively few studies concerning the long-term effect of strengthening method with the steel plate and prestressing wire on the confinement of extension of cracks, and consequently on the improvement of durability of concrete piers (Kojima et al.2000).

The purpose of this study is to comparatively investigate the long-term effect of the applied rehabilitation method in controlling the expansion of concrete due to ASR when the columns were exposed outdoors for about 5 years. Practical implications with regard to the rehabilitation for ASR affected concrete piers were also discussed.

## 2 EXPERIMENTAL PROCEDURES

### 2.1 Mix Proportions of Concrete and Dimension of Column

The cement was the ordinary Portland cement with the equivalent $Na_2O$ of 0.68 %. The andesite crushed stone from the Noto Peninsula in Ishikawa Prefecture was used as a reactive coarse aggregate; the sand and the gravel from the Hayatsuki River as a non-reactive fine and coarse aggregate, respectively. The evaluation of alkali reactivity of the andesite crushed stone used according to JIS A 5308 was not innocuous; the soluble silica content (Sc) and the reduction in alkalinity (Rc) at the chemical method: 228 m mol/l and 131 m mol/l, respectively, the expansion ratio at 6 months at the mortar bar test: 0.11 %. The main reactive components identified in the texture of the andesite crushed stone were the volcanic glass and cristobalite. Mix proportions of concrete are presented in Table 1.

**Table1 Mix Proportions of concrete**

| G. max (mm) | Slump (cm) | Air (%) | S/A (%) | W/C (%) | Cement (kg/m$^3$) | Water (kg/m$^3$) | Fine Aggregate (kg/m$^3$) | Non-Reactive Aggregate (kg/m$^3$) | Reactive Aggregate (kg/m$^3$) | NaOH (kg/m$^3$) |
|---|---|---|---|---|---|---|---|---|---|---|
| 20 | 8±2 | 2±1 | 42 | 53 | 308 | 164 | 784 | 562 | 563 | 7.54 |

Fig.1 shows the dimension of circle-shaped reinforced concrete column in the outdoor exposure test, in which the column was 800 mm in diameter and 1500 mm high. The D22 mm deformed bar and D16 mm deformed bar (SD 295A) were used as the axial and hoop steel reinforcement of the column, respectively. Carlson-type strain gauges and thermocouples were buried in the column when placing the concrete, and after demolding contact gauge tips with the distance of 100 mm were attached on the surface of the column. Also, the strain gauges with the length of 5 mm were stick on the surface of the steel reinforcement and steel plate in order to monitor the change in tensile strain. The data on the strain of concrete and steel reinforcement were automatically collected every week through the data logging computer system.

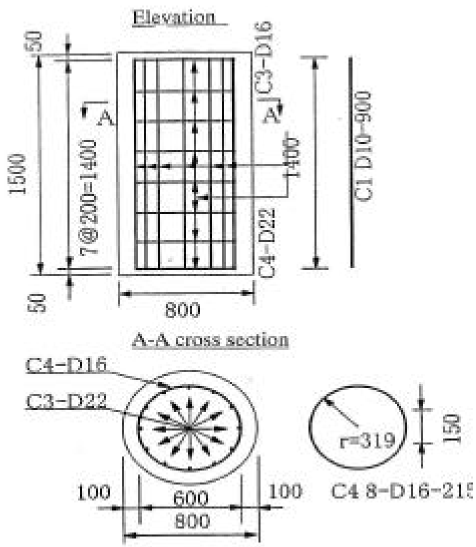
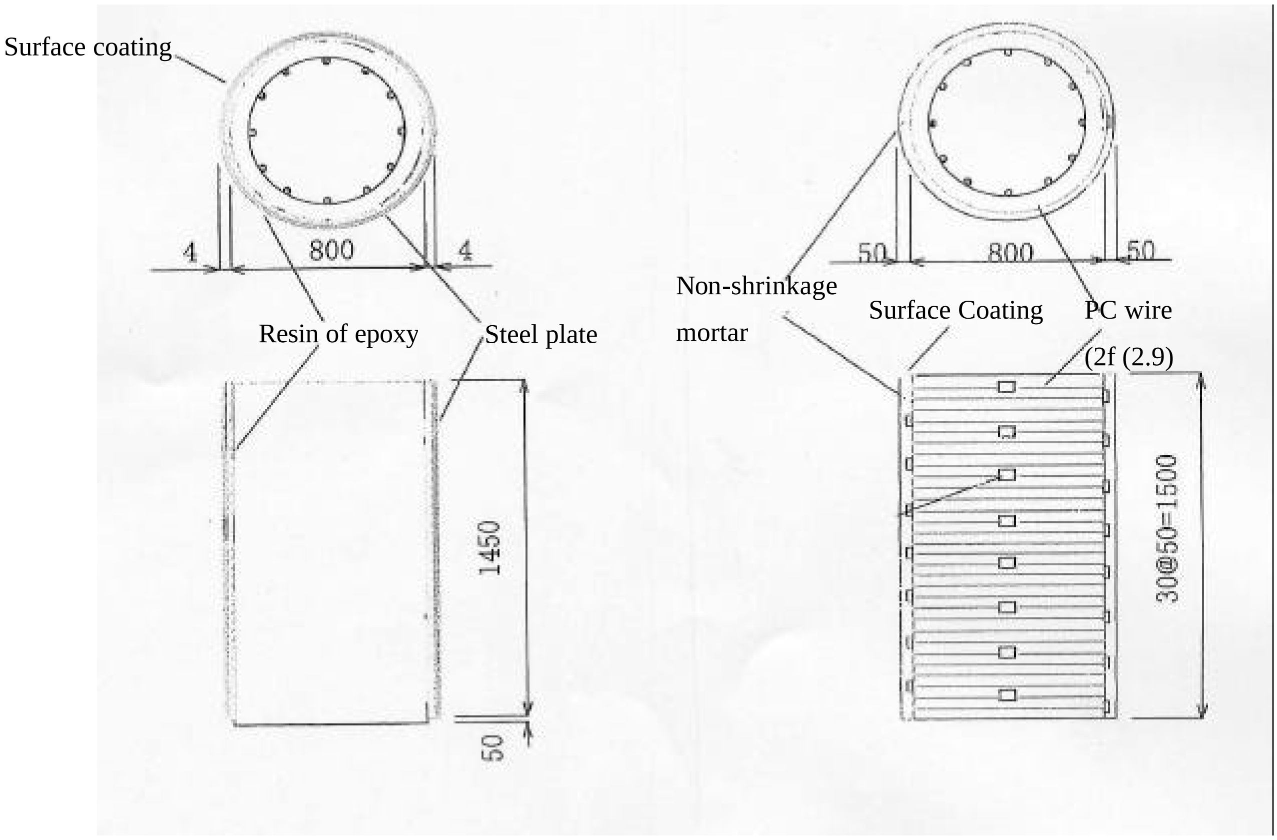



**Fig.1 Dimensions of circle-shaped reinforced concrete column**

## 2.2 Exposure Condition and Rehabilitation Method

The column was demolded at 3 days after placing the concrete, and followed by the steam-curing at the condition of maximum temperature of 60  C for 1 month in order to accelerate the expansion of concrete due to ASR. After the initial curing, the four columns were exposed outdoors at the campus of Kanazawa University for about 1 year until the width of cracks along the longitudinal steel reinforcement of the column had attained about 0.5 mm.

For ASR affected concrete columns, three types of rehabilitation methods, surface coating, steel plate bonding and prestessing steel wire confining, were applied except for one column as the controlling column. Fig.2 shows the schematic diagram of strengthening method with steel plate and prestressing steel wire. In the repair method by surface coating, all surfaces except the bottom one were coated with a highly elastic acrylic rubber type material with the overall thickness of 1 mm (Surface Coating, SC). On the other hand, in the strengthening method, one column was tied up and welded by the steel plate with a thickness of 9 mm, and the space between the column and the steel plate was injected with an epoxy-type resin (Steel Plate Bonding method, SPB); the other column was wound up with the prestressing wire of 2.9 mm in diameter at the interval of 75 mm, where the initial tensile stress induced was 20 % of its yield strength (Prestressing Steel Confining method, PSC). After repairing or strengthening, all columns were again exposed outdoors, and the beneficial effect of confinement of extension of cracks due to ASR was investigated for 4 years, as shown in Fig.3.




(i) Steel plate bonding (ii) PC confined methoid

**Fig.2 Schematic diagram of strengthening methods**

**Fig.3 Overview of exposure site of columns**

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## 3 RESULTS AND DISCUSSION

### 3.1 Expansion Behavior of Concrete in Column with and without Surfoce Coating

Fig.4 shows expansion curves of concrete on the surface of the column with and without the surface coating. It was observed that both columns with and without the surface coating expanded along with the seasonal change immediately after exposure outdoors; the concrete rapidly expanded during the summer, and it almost stopped during the winter. In the column without surface coating, the visible cracks occurred most remarkably along the trace of rainfall in the upper portion of the column at the south side. This shows that the degree of ASR may be significantly influenced by the microclimate condition around the column; the variations in temperature and moisture by the sunshine and rainfall. As shown in Fig.4, the expansion percentage of concrete was as large as 0.8 %, and the width of crack on the surface was 1 to 2 mm at average. On the other hand, the overall expansion of the column with the surface coating was smaller compared with the controlling one, but the effect of surface coating in controlling the expansion of concrete was not so significant as expected. It is considered that ASR is still progressing by using a relatively small amount of water in the column itself even when the supply of water from the environment has been completely shut out (Kawamura et al.1992). Fig.5 shows expansion curves of concrete in the center of the column with and without surface coating when exposed outdoors. As a whole, the expansion behaviors of concrete in the center of column measured by the Carlson-type strain gauge were not different from those on the surface. Interestingly, the strain of concrete at the horizontal direction was twice or triple larger than that at vertical direction, which was as large as 7000.. This may be attributable to both the dimension of circle-shaped column and the confinement ratio by axial and hoop steel reinforcement.

Fig.6 shows changes in the tensile strain of axial and hoop steel reinforcement with the exposure time. Especially in the column without the surface coating after 5 years of exposure time, the tensile strain of hoop steel reinforcement was about

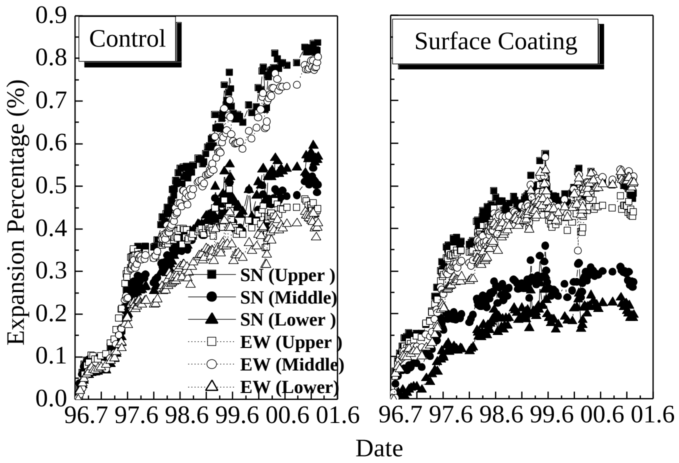


**Fig.4 Expansion curves of concrete on surface of column**

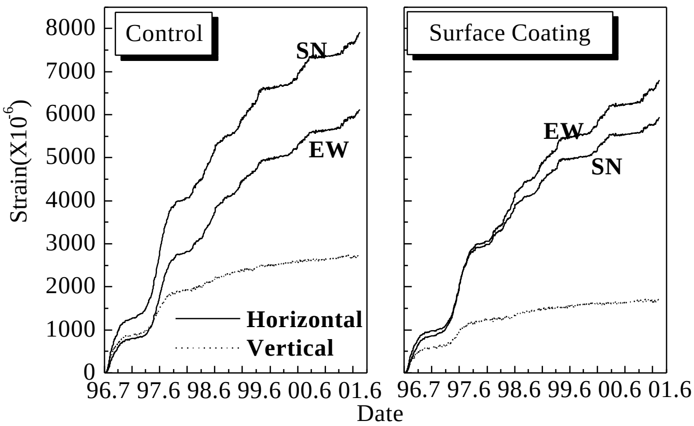


**Fig.5 Expansion curves of concrete in center of column**

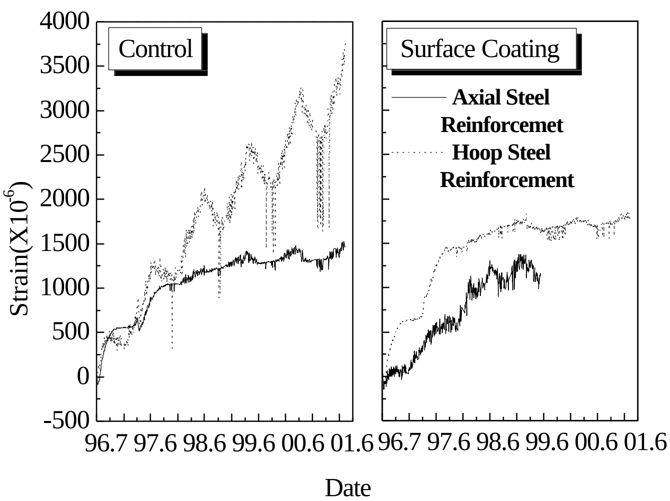


**Fig.6 Changes in tensile strain of steel reinforcement**

3500., which corresponded to a relatively large amount of expansion of concrete, although the tensile strain at axial steel reinforcement was about 1500., and it was almost constant after 2years of exposure time. This result indicates that the hoop steel reinforcement has already yielded partially and lost its effect in confining the concrete. Fig.7 shows the relations in the strain of concrete measured between on the surface and in the center of the column with and without the surface coating. It is found that at the early stage of exposure time, the strain on the surface is well equivalent to that in the center, but that after 2 years of exposure time, the latter is more than the former due to the decrease of confinement effect of concrete caused by the yielding of hoop steel reinforcement.

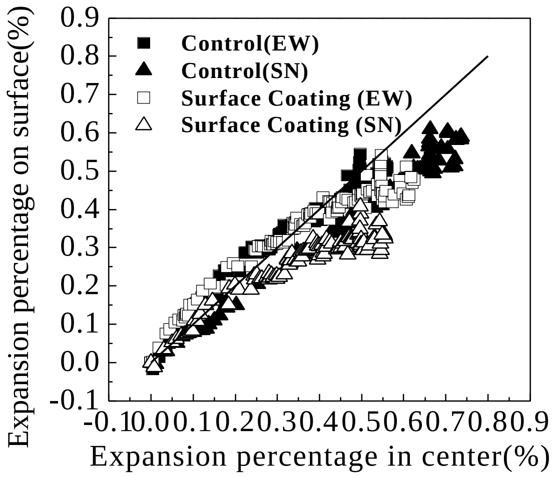


**Fig.7 Relations in strain of concrete measured on surface and in center of column**

### 3.2 Expansion Behavior of Concrete in Column Strengthened

Generally, it is considered that the load bearing ability of ASR affected concrete piers does not remarkably decrease. However, the recent field inspection work has shown that the cracks not only remain on the surface of cover, but also extend to the interior when ASR continues for a long period (Ono et al.2000). In serious cases, the hoop steel reinforcement or stirrup is also broken at the corner due to the excessive expansion of concrete (Torii et al.2001). A further research on strengthening such a seriously damaged concrete structure is needed.

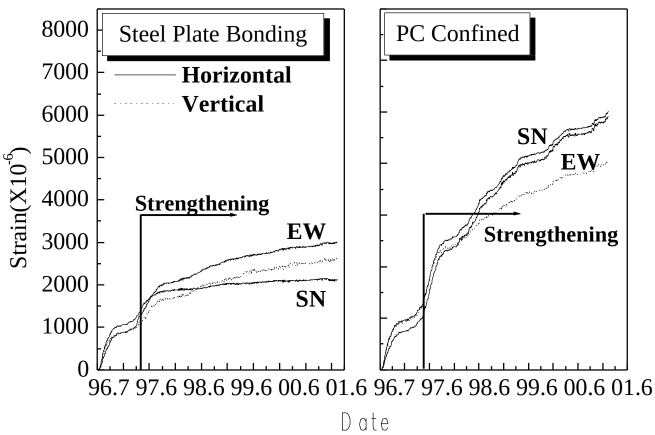


**Fig.8 Expansion cueves of concrete in center of column**

Fig.8 shows expansion curves of concrete in the center of the column strengthened with steel plate bonding and prestressing steel wire. As shown in Fig.8, strengthening methods with steel plate bonding and prestressing steel wire were effective in controlling the expansion of concrete. Comparing the effect of confinement of concrete by strengthening, the steel plate bonding method was more effective than PC confined method. In the steel plate bonding method, the strain of concrete in the center was as small as 30 % to 40 % that of the controlling one. Furthermore, by strengthening, the expansion curves of concrete became almost the same independently of the vertical and horizontal direction. When calculating the strengthening effect of steel plate and prestressing

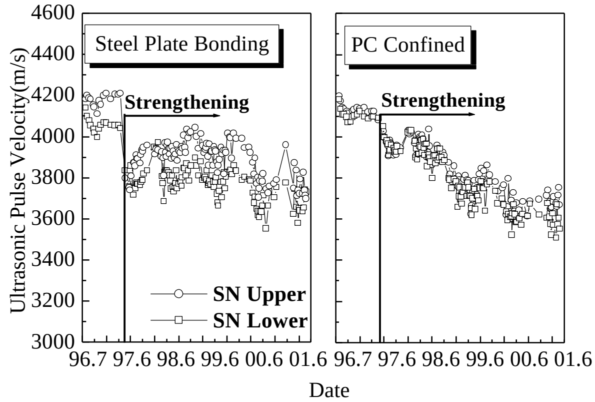


**Fig.9 Changes in ultrasonic pulse velocity of column**

steel wire in terms of the volume ratio of hoop steel reinforcement, its effect by the steel plate was six times larger than that by the prestressing steel wire. Fig.9 shows changes in the ultrasonic pulse velocity for the column strengthened with steel plate and prestressing steel wire. After strengthening, the ultrasonic pulse velocity of the column strengthened with steel plate showed the constant value of 3800 m/sec for a long period, although that of the column strengthened with the prestressing steel wire as well as the controlling one gradually decreased with the exposure time. The measurement of ultrasonic pulse velocity was suitable for estimating the degree of extension of cracks in the ASR affected concrete column. It is concluded that the load bearing ability as well as durability of concrete column strengthening with the steel plate or prestressing steel wire can be greatly improved by confining the extension of cracks due to ASR.

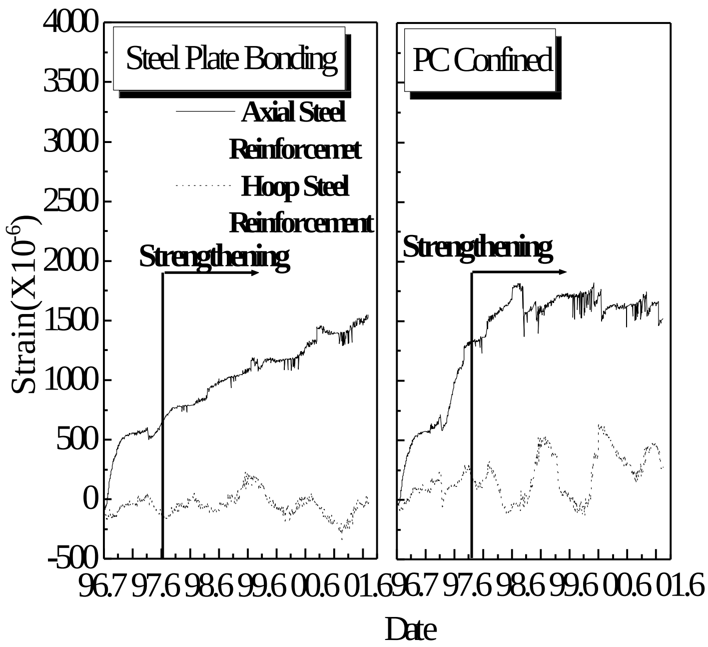


**Fig.10 Changes in tensile strain of steel reinforcement**

Fig.10 shows changes in the tensile strain of axial and hoop steel reinforcement of the column strengthened with steel plate and prestressing wire with the exposure time. In the column strengthened with the steel plate, the tensile strain of the hoop steel reinforcement was very small compared with the controlling one because the stress caused by ASR effectively transmitted to the steel plate in stead of the hoop steel reinforcement. However, the tensile strain of axial steel reinforcement was not at all decreased by the Poisson effect of concrete resulting from the confinement at the horizontal direction. Similarly, in the column strengthened with the prestressing steel wire, the tensile strain of the hoop steel reinforcement was reduced to a significant extent.

Strain Behavior of Steel Plate and Prestressing Steel Wire

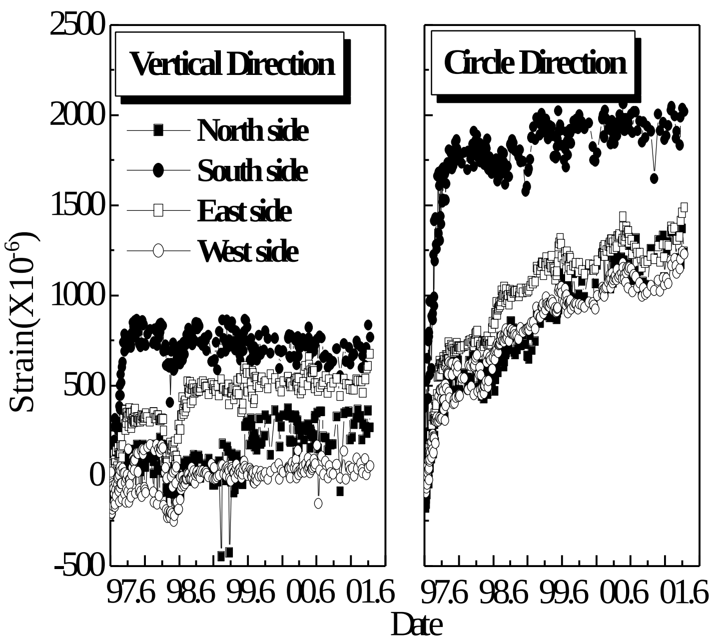


**Fig.11 Changes in tensile strain of steel plate**

Fig.11 shows changes in the tensile strain stick on the surface of the steel plate with the exposure time. The tensile strain behavior of the steel plate significantly varied depending on the side of column. That is to say, the tensile strain of steel plate at the south side was twice that at other side, because the increase in temperature of steel plate by the sunshine is the

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largest at the south side during the summer. Taking into considerations that the tensile strain of steel plate is more than 2000.m in the circle direction, which exceeds the strain of yielding of steel plate, the steel plate itself will break out especially at the portion of welding of steel plates when the expansion of concrete continues in the near future.

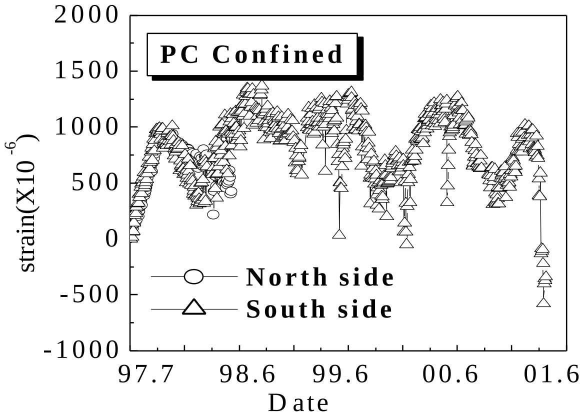


**Fig.12 Changes in tensile of prestressing steel wire**

Fig.12 shows changes in the tensile strain stick on the surface of the prestressing steel wire with the exposure time. Some gauges could not be measured at the early stage of exposure time, but sound gauges showed typical cyclic changes in the tensile strain between 1200. in summer and 500. in winter. It was found that the tensile strain of prestressing steel wire varied depending on the change in temperature. This shows that the stress induced by the prestressing steel wire also varies because the column itself will expand or shrink with the change in temperature to a lesser extent.

## 4 CONCLUSIONS

From the experimental results of long-term outdoor exposure test for ASR affected reinforced concrete columns, the strengthening method by steel plate and prestressing steel wire was effective in confining the extension of cracks due to ASR and in increasing its durability of concrete.

The following information was obtained in this study;

1. The expansion of concrete in column due to ASR continued for a long period, consequently the strain of concrete was 7000μ, and the hoop steel reinforcement yielded partially.
2. The effect of surface coating in controlling the expansion of concrete was not so significant in the relatively large-sized column used in the exposure test.
3. The strengthening method with steel plate was more effective in confining the extension of cracks due to ASR than that with prestressing steel wire.
4. The measurement of ultrasonic pulse velocity was suitable for estimating the degree of extension of cracks of the column.
5. The tensile strain behavior of steel plate in circle direction varied significantly depending on the south and the other side, possibly leading to breaking out at the welding of steel plates.

The monitoring on the strain of concrete and steel reinforcement in the column is still ongoing. At the final stage of outdoor exposure test, after taking off the surface coating and steel plate of the column, the deterioration of concrete on surface and in interior will be investigated in detail.

## 5 ACKNOWLEDGMENTS

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# Evaluation Of Accelerated Chloride Ion Diffusion Test And Applicability Of Fick's Second Law

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**Summary: A global test program is set up to find the correlation between the standard diffusion test (ASTM C1202-97) and the accelerated Chloride Penetration Time (CPT) test. The purpose is to find a relation between the steady state and the dynamic diffusion coefficient. In this way only the standard test would be necessary to quantify both diffusion coefficients. However, the CPT test gives extra information about the processes at the reinforcement bar and cylindrical concrete samples and various concrete cover thicknesses can be used.**

**In this paper the results of the first part of the CPT test program are reported. Two concrete mixes with a different porosity are tested. The CPT test is accelerated by a constant potential difference in a 1.5 and 3 % NaCl-solution. Not only the current during the whole test, but also the total amount of charge passed through the samples and the concentration of chloride ions in the concrete at different depths are investigated.**

**Two concrete mixes with different porosity are compared in the CPT-test at 1.5 and 3 % NaCl and at different applied voltages. The applicability of the test method is questioned and suggestions for further investigations are made.**

**Keywords: Accelerated chloride penetration time test, chloride profile, total amount of electrical charge, diffusion coefficient**

## 1 INTRODUCTION

One of the main durability problems of concrete is the corrosion of the embedded reinforcement. If the concrete cover on the reinforcements is thick enough, there will be no damage thanks to the alkalinity of the concrete. However, due to carbonation or chloride attack the protection of the reinforcement can be broken down. In this paper only the influence of chlorides will be investigated. Chloride ion ingress is a suction and diffusion phenomenon, which normally takes a long time. This time span must be shortened for control and design purposes, and therefore a number of accelerated chloride ingress test methods are proposed in literature, e.g. Andrade (1968) and Justnes (1990). However, the evaluation of the accelerated test results, their reliability and the transfer of these results to real life behaviour remains uncertain. As a consequence, several accelerated test procedures are used in practice.

A correlation is searched between the standardised ASTM C1202 diffusion cell test method and the accelerated chloride penetration time (CPT) test. The purpose is to derive a relation between the diffusion coefficient found in the standard diffusion test and the diffusion coefficient found in the CPT test. This would allow one to relate the accelerated tests to real life behaviour.

The first step within this investigation is a detailed study of the transport of chlorides during the CPT test. To get a better view on the mechanism of chloride ion transportation during the accelerated chloride penetration test, two concrete mixes with different porosity are tested. The effect of different voltages (3, 5 and 7 V) is examined by comparing the CPTs and the total amount of electrical charge that has passed through the sample during the tests. The chloride concentration profile is determined at the end of the tests.

## 2 ACCELERATED CHLORIDE PENETRATION TIME TEST

Chloride penetration in the concrete is generated by electrochemical processes in the accelerated chloride penetration time test. The migration test is executed on cylindrical samples. The samples are placed in a 3 % (or 1.5%) NaCl-solution, in such a way that the distance between the surface of the water/salt-solution and the top of the specimen is at least 20 mm (Fig. 1).

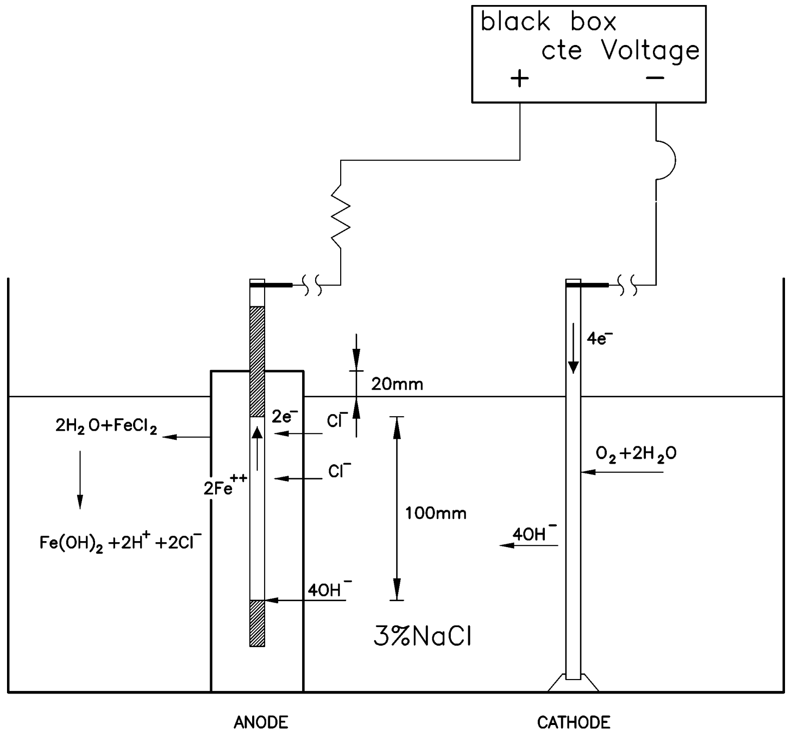


**Figure 1: Schematic representation of the electrical circuit of the test**

The 3% NaCl-solution corresponds approximately to the chloride concentration in seawater. A constant voltage difference of respectively 3, 5 and 7 V is applied between the reinforcement bar in the test sample and a stainless steel cathodic plate in the NaCl-solution. The reinforcement bar in the concrete acts as an anode (positive) in the system and consequently attracts anions ($OH^-$ and $Cl^-$). The stainless steel plate is the cathode (negative) of the system. At this plate the reduction of the oxygen ($O_2$) takes place ($4e^- + O_2 + 2H_2O \rightarrow 2OH^-$). The hydroxyl ions move towards the anode together with the chloride ions present in the water. The electrolytic medium consists of the NaCl-solution and the pore solution in concrete. A process takes place similar to non-accelerated electrochemical corrosion of steel.

The $Cl^-$ ions break though the passive oxide layer on the reinforcement, and attack the reinforcing bar. This results in pitting corrosion of the reinforcement bar.

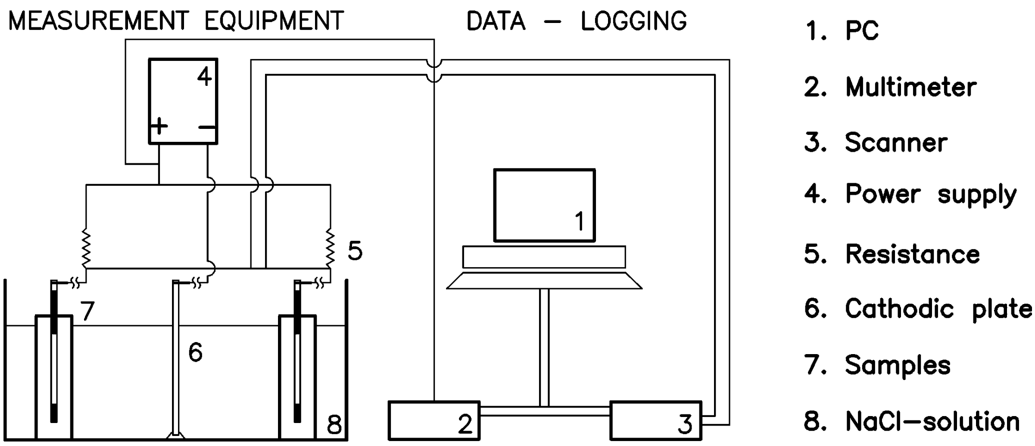


**Figure 2: Schematic plan of the test setup of the accelerated CPT-test**

The test setup is presented in figuur 2. The electrical current through the system is automatically recorded. Therefor, the voltage drop over a 1 Ohm reference resistance is measured and recorded. According to Ohm’s law this voltage drop is converted to a current: V = R.I = 1.I

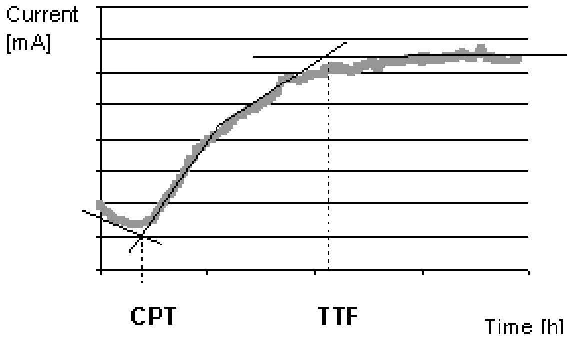


**Figure 3: Theoretical evolution of the current during a CPT-test.**

This current is an indication for the corrosion rate. The expected time curve of this current is represented in figure 3. Four stages can be distinguished.

1. In the first part a current decrease is caused by polarisation of the anode. The $OH^-$-ions strengthen the passive oxide layer. These ions are already present in the pore-water long before the chloride ions and they too are negatively charged. The strengthening of this patina makes that less electrons are released and that the current decreases.
2. In a second stage the current increases. This is the moment the chloride ions reach the steel bar in the concrete and start the corrosion. This point of time is called the chloride penetration time or CPT. Shortly after reaching the CPT time cracks will appear in the concrete. Due to the fact that the potential difference between steel and water is now imposed to a much higher level than in natural conditions of corrosion, the concentration of chlorides necessary for corrosion is much lower. Consequently, the corrosion starts at the moment the first chlorides reach the reinforcement bar (CPT).
3. From that moment on the patina becomes weaker and cracks become bigger. Consequently, the chloride ions have better access to the steel and the corrosion process is accelerated.
4. In a final stage the samples totally break. This is visible by a vertical crack all over the specimen. The corrosion products flow out off the concrete and silt up the fissure. The current reaches a maximum value. This is characterised by the time to failure or TTF. Due to the silting process the current remains about constant.

The CPT is derived from the current evolution curve at the intersection of two lines tangent to the descending and ascending part of the curve.

## 3 EXPERIMENTAL DETAILS

### 3.1 Materials

Two different concrete mixes with different porosity are used. Both are made with a Portland cement CEM I, 42.5, according to the standard EN 197-1. The composition of the mixes is given in table 1.

**Table 1: Composition of the two concrete mixes.**

| | Composition 1 | Composition 2 |
|---|---|---|
| Coarse aggregate 4/14 | 1119 kg/ $m^3$ | 1003 kg/ $m^3$ |
| Fine aggregate 0/5 | 699 kg/ $m^3$ | 896 kg/ $m^3$ |
| CEM I, 42.5 | 300 kg/ $m^3$ | 291 kg/ $m^3$ |
| Water | 180 kg/ $m^3$ | 180 kg/ $m^3$ |
| w/c-ratio | 0.62 | 0.60 |
| porosity | 18-20 vol.% | 12-15 vol.% |

The 28-day compressive strength of both mixes is 39.4 respectively 45.4 MPa.

The original chloride content of the two concrete mixes is negligible. The values are given in table 2. Because no chlorides are added to the mix and the aggregates are free from chloride contamination, the only source of chlorides is the tap water used for mixing. This water has a chloride content of 0.047 g/l.

**Table 2: Concentration of chloride in the concrete before the experiments**

| | Mix 1 | | Mix 2 | |
|---|---|---|---|---|
| | w% Cl water soluble (potentiometric) | w% Cl acid soluble (NBN B15-250) | w% Cl water soluble (potentiometric) | w% Cl acid soluble (NBN B15-250) |
| Top | - | 0.001 | 0.010 | 0.016 |
| Centre | - | 0.006 | 0.011 | 0.019 |
| Bottom | - | 0.015 | 0.011 | 0.013 |

### 3.2 Test specimens

The dimensions of the specimens are given in figure 4. It concerns cylindrical concrete samples with a smooth drawn steel bar (ST52) in the centre. The bar has a diameter of 8 mm and is placed 25 mm above the bottom of the sample. The concrete cover of the bar is about 21 mm. After sandblasting, the top and the bottom of the steel bars are covered with epoxy. Only the remaining 100 mm of untreated steel can be attacked by chloride ions. Another important reason for this treatment is to prevent oxidation corrosion at the concrete-steel-air interface.

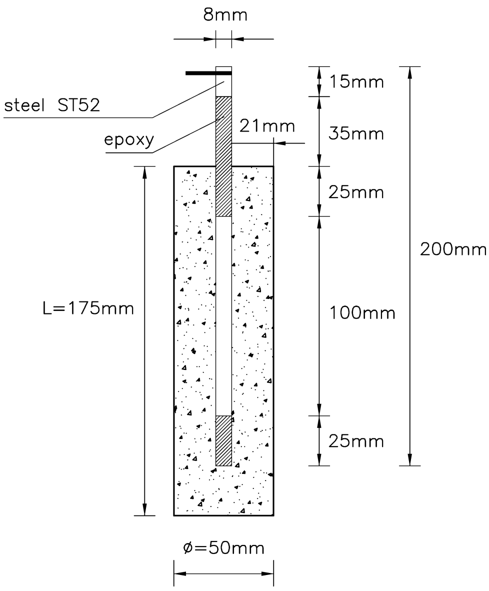


**Figure 4: Schematic representation of the cylindrical specimens**

The test samples are cored out of a concrete plate with a thikness of 175 mm. One day after pouring the concrete, the formwork is removed and the concrete plate is cured for 28 days in water at 20°C. Then the cylinders are cored out of the concrete plate. Subsequently, the cores are saturated for 7 days. In this way capillary suction of water during the first minutes of the test is prevented, and only diffusion of the $Cl^-$ is possible to penetrate into the concrete cover.

## 4 RESULTS AND DISCUSSION

### 4.1 Results

The results of the CPT test are compared according to the composition of the concrete and to the applied voltages. Firstly the focus is on the impact of the porosity on the CPT. Therefor three samples of mix number 1 and two samples of mix number 2 are submitted to a constant voltage of 3 V and a NaCl concentration of 3 %. Secondly the influence of different voltages has been examined, namely 5 and 7 V. In this case the concentration of NaCl is reduced to 1.5 %.

Figure 5 gives the current versus time graphs of the samples of mix 1. Three different voltages are applied each time on three samples. Based on these curves the CPT can be determined. As an illustration the determination of the CPT of a sample of mix number 1 tested with 7 Volt and in a NaCl-solution of 1.5 % is given in figure 6.

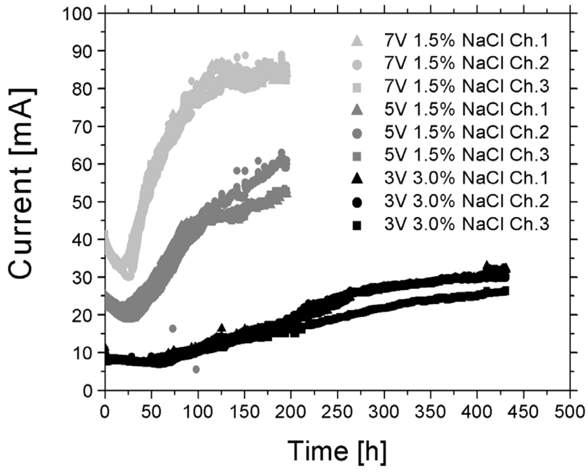


**Figure 5: Overview current trough samples during the CPT-test**

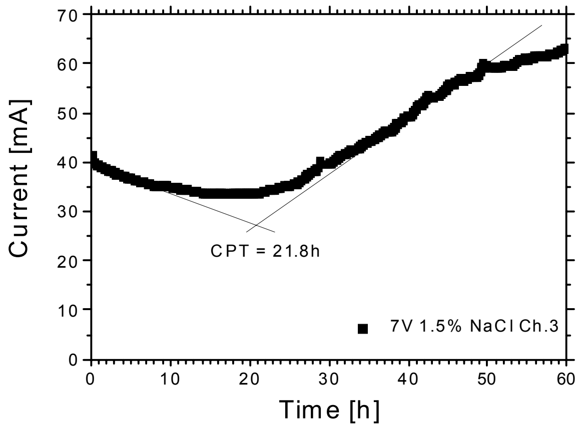


**Figure 6: Graphical determination of the CPT**

The CPTs of the tested cylinders are summarised in table 3.

**Table 3: Overview of the CPT (Chloride Penetration Time)**

| Samples | Individual CPT [hours] | Mean CPT [hours] | Standard deviation [hours] |
|---|---|---|---|
| Comparison between mix 1 and 2 | | | |
| CPT mix 1/ 3V/ 3.0 %-NaCl | 38.8, 42.6, 59.0 | 46.8 | 10.7 |
| CPT mix 2/ 3V/ 3.0 %-NaCl | 89.2, 85.8 | 87.5 | 2.4 |
| Comparison between 5 and 7V | | | |
| CPT mix 1/ 5V/ 1.5 %-NaCl | 24.0, 29.2, 30.5 | 27.9 | 3.4 |
| CPT mix 1/ 7V/ 1.5 %-NaCl | 18.1, 27.1, 20.1 | 21.8 | 4.7 |

Another indication for the corrosion is the total amount of charge passed through the samples during the test, because each Fe-atom that splits releases not only a $Fe^{2+}$ but also 2 electrons. This value can be obtained by integrating (Fig. 6) the current versus time curves. It is a measure of the electrical conductance of the concrete during the period of testing. Although the testing period was not always the same, it is interesting to compare the relative magnitudes of the total electrical charges.

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**Table 4: Overview of the charge passed through the samples**

| Samples | Time [h] | Individual electrical charge [C] | Mean electrical charge [C] | Spreading [C] |
|---|---|---|---|---|
| Comparison between mix 1 and 2 | | | | |
| CPT mix 1/ 3V/ 3.0 %-NaCl | 331 | 30302, 30487, 25964 | 28918 | 2560 |
| CPT mix 2/ 3V/ 3.0 %-NaCl | 402 | 19814, (22975, 24520), 18706 | 19260, (21504) | 783, (2705) |
| Comparison between 5 and 7V | | | | |
| CPT mix 1/ 5V/ 1.5 %-NaCl | 309 | 27973, 28605, 26151 | 27576 | 1274 |
| CPT mix 1/ 7V/ 1.5 %-NaCl | 262 | 48617, 46461, 48602 | 47893 | 1241 |

At the end of the CPT tests, slices were cut from the samples to determine the chloride concentrations profile. Two slices were cut from the samples of the first mix (5V), at depths of 5 and 17 mm (Fig. 7). In the investigation of the second mix (3V), three thinner slices were analysed, namely at 4, 11 and 18 mm depth, to obtain a more detailed chloride penetration profile (Fig. 7).

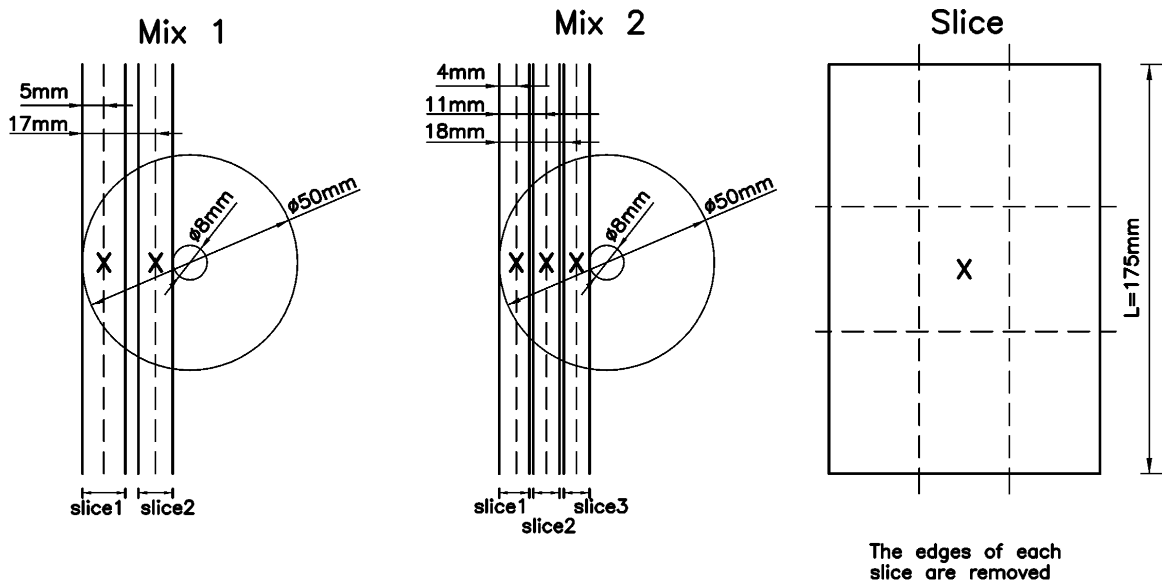


**Figure 7: Position of $Cl^-$-concentration sampling**

Only the free chloride ions can contribute to the corrosion of the steel bars. In the framework of this investigation the water-soluble and the acid soluble chloride ions are determined (Fig. 8). Neither of the two concentrations really represents the amount of free chloride ions, but they give an estimation of the amount of available $Cl^-$ ions in the process. Even the amount of water-soluble chlorides includes dissolved chlorides present in the form of crystalline salts. For the second mix only this amount of the water-soluble chlorides is determined (Fig. 8).

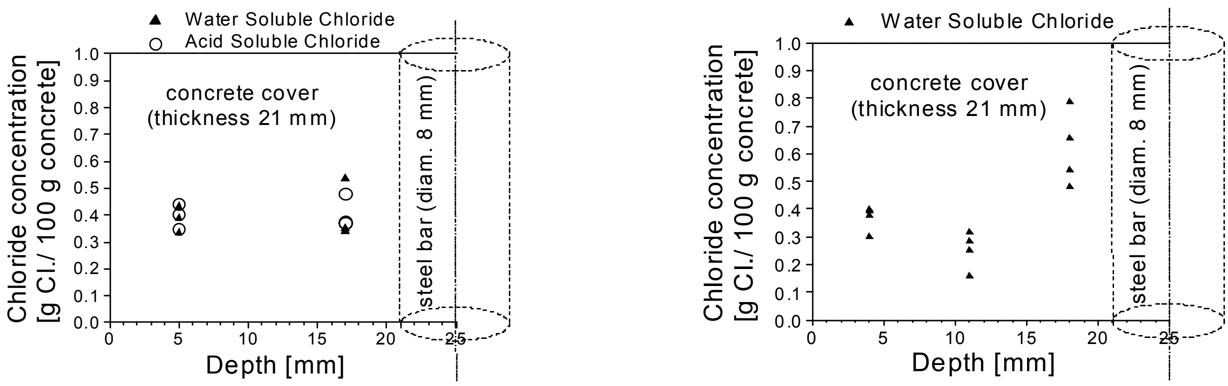


**Figure 8: Chloride concentration at different depths in the concrete mixes 1 and 2 after CPT tests**

### 4.2 Discussion

#### 4.2.1 Chloride Penetration Time

No general conclusions can yet be made after this first part of the global test program. Comparing the CPTs of mix number 1 and 2, it can be seen that the porosity has a great influence on the penetration time of the chlorides. The more porous the concrete, the lower the CPT value. It takes almost twice as long to penetrate trough the less porous concrete. In a porous water saturated environment the resistance against the transport of chloride ions is low. This confirms the general statement that the concrete cover has to be of a high quality to provide adequate protection of the reinforcement. Not only it has to be of an adequate thickness, also the permeability should be watched.

Although the salt concentration in the tank is only 1.5 %, instead of 3 %, in the second comparative investigation, the CPTs here are much lower. This means that lesser chloride ions result in a faster degradation. This could mean that the accelerating effect of the potential difference is very big. On the other hand it might not be justifiable to use excessive potential differences up to 7V. The initial intention was to accelerate the process because normal degradation processes of steel bars in concrete take many years. But, is the influence of the voltages limited to the acceleration? It should be investigated if the high voltages do not ionise the water molecules, because the normal redox potential of water is 1.23V ($2\ H_2O \Leftrightarrow O_2 + 4H^+ + 4e^-$) (Polytechnic pocketbook 1995). This dissolution would totally change the test conditions. In that case the measured currents are not resulting from chloride diffusion only, but also from the $OH^-$ ions ($2\ H_2O + 2\ e^-\ . \Leftrightarrow H_2 + 2\ OH^-$).

Parallel real time tests are necessary to find out the acceleration rate, which would allow the calibration the test results to real life situations. After calibration the diffusion coefficients of the CPT tests can be used for design purposes.

#### 4.2.2 Total amount of electrical charge passed through the samples

The standard ASTM C1202-97 gives a indication table for the chloride ion penetration based on charge passed during 6 hours. For the CPT test no tables are available yet, but based on the same principle it should be possible to define certain limits. The results in table 4 show that there is a correlation between the diffusion possibility and the total amount of charge. The more porous concrete (mix 1) and therefor also more open for chloride ions gives a higher charge than mix 2. Nevertheless, the results coming from the other applied voltages demonstrate that the eventually established reference table should be adapted for different applied voltages. The higher voltages (5 and 7 V) accelerate the transport process, and thus more ions are transported in a shorter time through the same concrete with the same electrical conductivity.

#### 4.2.3 Chloride concentration in the concrete

The samples of the first mix tested in 1.5 % NaCl solution and with a voltage of 5 V, show a very slightly increasing chloride concentration in function of depth. In a second phase, where more information is gained from the samples of the second mix (with 3 % NaCl and 3 V), the chloride concentration decreases from the concrete surface to the inside and than increases again at the steel bar.

Knowing that the samples were saturated, capillary suction is out of the question. Also convection can be eliminated, because the transport occurs inside the concrete. Diffusion seems a possible way of transport. In theory, the laws of Fick can describe the diffusion of chloride ions through concrete. Fick's second law describes the process in a non-steady stage. Consequently, the idea is to apply an adapted version of the second law where the influence of the imposed electric field would be taken into account. In literature many suggestions are made for such adaptation. Andrade et al (1994) proposed the following expression:

$$C_x = C_s\left(1 - erf\,\frac{x}{2\sqrt{\frac{ZF\Delta E}{RT}D_{app}t}}\right)$$

with $C_x$ = concentration at depth x [kg/m$^3$]

$C_s$ = concentration at the surface [kg/m$^3$]

$D_{app}$ = diffusion coefficient [m$^2$/s]

Erf = error function

Z = electrical charge of species [equivalent/mol] ($z_{Cl} = -1$)

F = Faraday number = 96.500 [C/ equivalent]

R = gas constant = 8.314 [J/mol.K]

T = absolute temperature [K]

However, it has to be remarked that this law is mostly used for diffusion of chloride ions through a concrete plate which separates one fluid from another one. The circumstances of the CPT test are different. Because of the treatment with epoxy (Fig. 1, 4) one can consider the problem as circular cylindrical. Consequently, the concentration is only a function of radius r and time t. The diffusion equation becomes:

$$\frac{\partial C}{\partial t} = \frac{1}{r}\frac{\partial}{\partial r}\left( rD\frac{\partial C}{\partial r} \right)$$

If the diffusion coefficient is considered constant, it can be put in front of the equation. The solution of this expression is determined by both the initial and the boundary conditions of time and place, which determine the specific character of the diffusion process.

The initial concentration of chlorides in the concrete of the tested samples is known (table 1) as well as the surface concentration.

$C = C_0 = 1.5$ or 3 % NaCl (9 or 18 g Cl/l), $r = 25$ mm, t/0,

$C = f(r) = C1$ (table 1), $0 < r < a$, $t = 0$,

Because the concentration is initially uniform throughout the cylinder, the solution can be reduced to:

$$\frac{C - C_1}{C_0 - C_1} = 1 - \frac{2}{a}\sum_{n=1}^{\infty}\frac{\exp(-D\alpha_n^2 t)J_0(r\alpha_n)}{\alpha_n J_1(a\alpha_n)}$$

$J_n$ = Bessel function of the first kind of order n

$\alpha_n$ = solutions of $J_0(a\alpha_n) = 0$

From the above equation, different values for the diffusion coefficient were found in function of the depth. Also starting from a constant diffusion coefficient it was impossible to find the concentration profile given in figure 8. In the expected theoretical concentration profile corresponding to the law, the concentration of chlorides starts to grow from the surface towards the centre and increases in function of time until the concentration is constant everywhere.

In this case the law does not correspond to reality. The main reason for that is most probably the presence of cracks. Physical processes in cracks are totally different from processes in a homogenous medium. Therefor, the CPT test should be redone and the concentration of chlorides should be determined just before cracking.

Another reason is the presence of bounded chlorides. The concentration inside the concrete is higher than the enforced concentration at the surface. This confirms the fact that from the moment the first chloride ions diffuse into the concrete they are absorbed to the inner wall of the pores. It is known that the bonding capacity of concrete for chlorides is rather high.

Tang and Nilsson (1996) assumed that chloride binding is dominated by adsorption on the CSH-gel. It is also found that the ratio between free and bounded chlorides is not independent of the chloride concentration, nor for chemical binding (e.g. Friedel salt formation) nor for adsorption. Consequently, it is impossible to use a constant factor to determine the free chlorides on the basic of the measured bounded ones. Furthermore, McGrath and Hooton (1996) claim that chemical binding will slow down the “break through” time and thus reduce the diffusion coefficient. Probably, the chloride ions entered in the surface layers are immediately absorbed due to the presence of positively charged particles on the concrete inner surface of the pores. The chlorides will only start to move if other chlorides or aggressive elements, like sulphates, already occupied the bonding places. Besides, the binding activity was stimulated in our tests because of the young age of the used concrete.

It is not appropriate to attempt to calculate the diffusion coefficient by means of an adapted version of the second law of Fick, including the influence of the electric field, the bounding capacity and the time effect of the not yet mature concrete. As it is mentioned in the description of the test procedure, fissures occur. Together with these cracks, the overall structural composition changes. The chloride ions can easily reach the reinforcement bars through these open channels. The main adaptation to the equation should therefore be with respect to these structural changes.

## 5 CONCLUSIONS

The CPT test is an interesting test method to study the penetration of chlorides in saturated concrete samples. Therefore, this test has been evaluated as a first phase of a global investigation.

The CPT test consists of different stages and a lot of parameters play a role. The electrical current gives information about the electrical charge that has passed through the sample. The first decreasing part can be explained by the attraction of $OH^-$-ions present in the concrete cover, which cause strengthening of the passive oxide layer of the reinforcement bars. The chlorides penetrate in the concrete, but are immediately adsorbed by the charged particles at the inner surface of the pores. Therefore, the chloride profiles obtained can not be used to determine the diffusion coefficient by means of the second law of Fick, even not after adapting to the electric field used and to the adsorption capacity of concrete. The presence of cracks makes it necessary to review the transport processes.

In order to use Fick's laws in the future, the concentrations of chlorides should be determined before cracks appear. This moment can be determined by means of the current time curve of the CPT test.

Together with the CPT tests the standard diffusion test can be executed on the same mixes to analyse the correlation between the diffusion coefficients.

The actual importance of the CPT test lies in its ability to allow quick comparative tests on different compositions and geometrical layouts of reinforced concrete. It can be used as a test method for hydrophobic agents on concrete. Changes in the diffusion coefficient and chloride penetration times can be used as process parameters. One of the most promising applications of this test method is the analysis of corrosion protective products, which penetrate through the concrete cover and fix onto the reinforcement bar itself. For such applications standard diffusion test methods fail.

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# The Systematic Approach Of An Experimental Program To Support Service Life Prediction: A 4 Year Experience On External Wall Building Component

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**Summary: In this paper is reported the preparatory work and the final experimental program of an investigation performed in order to study the decay of performances of some building components by means of laboratory and outdoor aging tests, according to the general approach designed by ISO 15686-1. This experimental investigation is part of a larger on going research program agreed between DISET and LTS (Technical Laboratory, Department of Construction and Territory, Switzerland) started in 1997 whose goal is the development of a methodology for the prediction of building components service life and for the modeling of degradation mechanisms. The aging tests have been performed on building components belonging to the class of the external walls. The first kind of samples tested, have been partial sample (PS) representing the external layers of an entire technical solution, composed of perforated brickwork coated with render and of a protective layer consisting of an acrilical water painting. For comparison, the aging tests have been also performed on a series of unprotected brickworks. The results of this comparison have been measured between the time behavior of different solution of protected sample and unprotected ones : this has given information about the degree of protection during time obtained by different solution of painting, and a fundamental reference for comparing new market solution used in building construction. The accelerated aging program, has been conducted within an environmental test chamber, developing on the iteration of a cycle consisting of four distinct phases in which are simulated the most significant climatic conditions and actions (rain, irradiance, freeze and thaw), to which the chosen building components are generally exposed. Parallely to the laboratory aging program, outdoor aging has been also carried out from 1998 on the same typology of partial samples (PS). The outdoor aging testing are being conducted in Italy and Switzerland to find out time re-scaling and this to obtain the real service life of the components, in function to the different climates and pollution conditions. In a next and final phase of this first test program on partial sample (PS), the experimentally collected data has been elaborated and interpreted in order to support and validate the proposed methodology for evaluating the service life of building components. The second phase of the research will study the whole solution (WS) in his complex configuration as used in constructions. The related experimental program, just started in march 2001, will consist in substituting the flapdoor of the aging chamber by a section of the real building component and the aging cycles repeated to simulate its behavior between the outdoor environment (aging chamber) and indoor environment (laboratory at controlled conditions). All this experimental data will be useful to support the original analytical material sensitivity evaluation method and to define a final durability and service life evaluation method.**

**Keywords: service life prediction, systematic methodology and experimental program, predictive models, degradation mechanisms, durability of construction components.**

## 1 SELECTING THE TECHNICAL SOLUTION AND SETTING UP THE EXPERIMENTAL PROGRAM

The first step of the experimental program has been to choose technical solutions available within the range to form the basis for starting the experimental phase.

The criteria used for this selection was to opt for a solution that is most widespread on the traditional building market, and which has a capacity, in terms of results and considerations, that is the most broad reaching and vast as possible, while being at the same time the most immediate to verify.

Therefore the starting point was the solution PV2 from the range available, comprising double cavity wall in masonry, with intermediate insulation, the surfaces of which are rendered, with a layer of protective paint applied on the external wall surface.

After choosing the technical solution for the basis, the experimental program could then be configured. The program was structured into three steps, which led to the same number of experimental variables:

The first choice was to perform parallel *accelerated* tests in conditioned laboratory environments and external *natural* tests, subsequently verifying the possibility of temporal re-scaling of results. Results from the accelerated tests must be validated by those read in the natural environment, and the compared results of the effects measured should lead to equivalent values between accelerated duration and effective duration.

The second step was to consider two types of sample: the *external version* of the selected technical solution, representing the layers that are directly subject to influence by external agents (Partial Sample – PS), and *entire solution*, which improves obtainable results, as it is also capable of simulating mechanisms and gradients found in reality (Whole Solution – WS).

Lastly the choice was made to consider the *degree of protection* obtainable with the paints as a variable of the system, and for this purpose it was decided to start from unprotected solutions leading on to measure relative categories of paint, varying the type of resin and weight percentage in the paint composition.

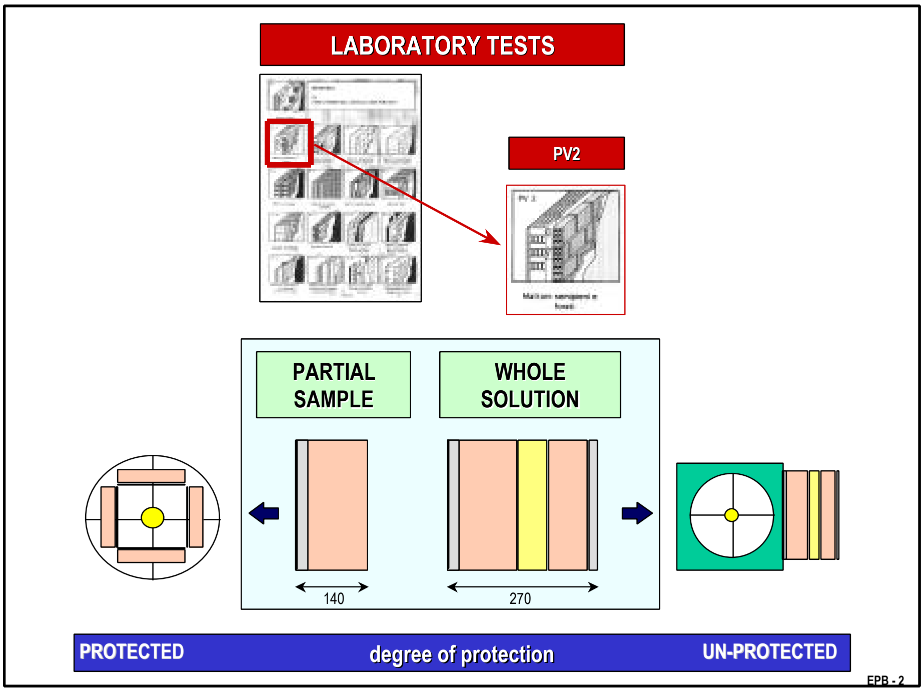


**Figure 1. Experimental program methodology**

## 2 CHARACTERISTICS AND COMPOSITION OF SAMPLES

When defining the criteria for the basis of selecting the materials in the technical solution subject to examination, special attention was paid to the need, throughout the experiment, to have materials with initial technical characteristics suited to the technical solution to be implemented and, above all with reproducible characteristics.

Therefore, before final selection, all components underwent suitability tests in laboratories. Components subject to precise quality requirements laid down by standards (such as cement) were also subject to conformity tests.

Furthermore, the composition of mortar for rendering was formulated taking into consideration directives regarding preparation of render for external use, issued by the Federal Laboratory for material testing and research of Dübendorf, Switzerland (EMPA, 1985). For example, on the basis of the suitability test results, it was decided not to use pre-mixed sand.

The following section lists the components and sub-products used to make the test samples, together with a description of their main characteristics.

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1. Bricks

As elements for the masonry work, perforated bricks were used with normal vertical perforations (code MS (A) 11-21) according to UNI 8942/1. In particular elements were used with the dimensions 12 x 12 x 25 cm with a perforation percentage of 41% and mechanical resistance to compression of 38 N/mm$^2$.

2. Sand.

As aggregate material for the mortar production, selected natural sand was used, with a particle composition obtained by mixing eight different fractions. The mix percentage of each fraction is illustrated in table 1 and figure 2.

**Table 1. Particle composition of sand**

| *filter* | *filler* | *0 - 0,20 mm* | *0,20 – 0,35* | *0,45 – 0,60 mm* |
|---|---|---|---|---|
| *% in mass* | 6 | 12 | 11 | 14 |
| *filter* | *0.60 – 1.00* | *1.00 – 1.50* | *1.50 – 2.50* | *3.00 – 4.00 mm* |
| *% in mass* | 12 | 15 | 19 | 11 |

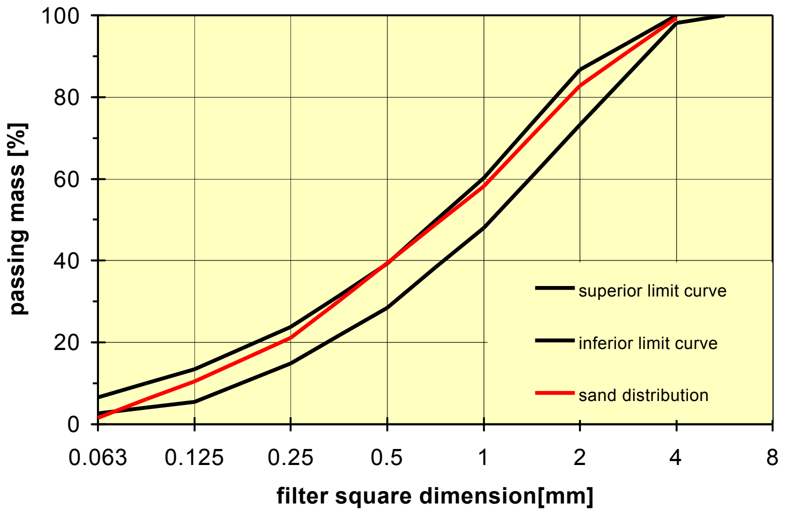


**Figure 2. Particle distribution of sand**

The mix percentage of the fractions was selected to obtain a uniform particle distribution, and was included in the grading envelope, defined on the basis of results of experimental research performed at the Federal Laboratory for material testing and research (EMPA) of Dübendorf : the grading envelope defines the zone within which the curve describing the distribution of sand grain particles needs to remain to be recommended for the grading envelope of render mortar.

The particle size analysis of some pre-mixed commercial sands has shown that the distribution of particle size of sand did not remain within this grading envelope. For this reason these types of sand were not deemed suitable for the production of render mortar and were therefore rejected.

The check of variability of the recipes on the final results of the solution lifetime (such as particle composition of the sands for rendering, type of masonry or insulation) was left for future experimental research.

3. Cement

The binder used for the preparation of both the mortar for the masonry and the rendering is identified according to the standard UNI ENV 197/1 as CEM II/A-L 32.5.

The cement used is a Portland lime cement comprising 80-94% clinker and 6-20% lime in the resistance class 32.5, i.e. with an initial normal compression resistance and a resistance to compression after 28 days greater than or equal to 32.5 N/mm$^2$.

4. Mortar for masonry

The recipe for the cement mix to be used as mortar for masonry was defined to satisfy a number of requirements regarding workability and resistance to compression.

The doses of the components used for preparation of the mortar are as follows: 1,450 g sand, 450 g cement and 250 g water.

5. Masonry

The masonry involved the use of stone elements as described in point 1, comprising a vertical structure with a thickness of 12 cm and vertical joints with mortar and continuous horizontal joints.

This type of structure features, in accordance with standard UNI 10355, a surface mass of 140 $kg/m^2$ and heat resistance of 0.24 $m^2K/W$.

The dimensions of the masonry were selected to be compatible with the space restrictions inside the artificial accelerated ageing equipment.

Each test sample comprises four courses of bricks each made up of 1.5 elements.

The surface of the test samples, including joints, extends over 0.21 $m^2$ (L x A = 39 cm x 53 cm). The volume of the samples, including a 15 mm layer of rendering, is approximately 35 kg.

6. Rendering mortar

As a binder for the preparation of the mortar, a mix was used comprising 70% cement type CEM II/A-L 32.5 and 30% highly hydraulic lime, with a normalised resistance to compression of 6 $N/mm^2$.

The doses of the components used for mortar preparation are as follows: 1,450 g sand, 315 g cement, 135 g hydraulic lime and 280 g water.

7. Paint coating product

The protective systems tested comprised four products belonging to continuous systems classified as follows: by standard UNI 8752 as applied coatings comprising an external surface film on the base of an appreciable thickness; by standard UNI 8681 as paints, with functions conforming to that of undercoats and finishing coat, comprising resins in water dispersions, one-pack coatings, with physical drying, a smooth opaque appearance, and decorative and protective properties specially required for all external paints, such as those considered in the program in question.

For a better interpretation of the behaviour on-site of a specific paint, its typical allocation must be considered, i.e. that it comprises temporary components with the sole function of rendering it fluid, more or less viscous, for subsequent application on a specific base, where it changes to a solid state and finally shows its actual properties.

In the fluid state, the main characteristics of a paint regard the stability of the container, the formation of an easily removable deposit even after months of storage, the lowest viscosity possible for application with a minimum dilution to avoid reducing the thickness of the solid layer, and optimal manageability. The performance characteristics of a paint depend on the solid state, comprising pigments – fillers (powders) and resin or binder, i.e. the property of remaining intact over time; a property due to the type of resin and its ratio, in volume, with the powders defined with the acronym CVP, (Concentrazione Volumetrica Polveri – Powder Volume Concentration), the value of which refers to the percentage contained in the solid coat.

The graph illustrates the variation in characteristics of all paints, water or solvent based according to variations in the CVP. The criteria for selection of the pains to be examined was based on products most commonly used on the market, both as type of resin and content of the latter.

### 1.1 Identification of the paints prepared for the current program

On the basis of these considerations, four types of water based paint were prepared for the current research program, with two types of resin, acrylic and versatic vinyl, and two different CVPs as shown below.

*Identification of paint with high resin content. UNI 8757*

Type of resin: non plastified acrylic and versatic vinyl paint, both in water dispersion

| | |
|---|---|
| Specific gravity: | 1,380 - 1400 g / litre |
| Solid content by volume: | 38 - 40 % |
| Solid content by mass at 105 °C: | 56 - 58 % |
| Solid content by mass at 450 °C: | 39 - 41 % |
| Calculated C V P: | 44 - 45 % |
| Calculated resin on dry coat at 105 °C: | 29 – 30 % |
| Colour: | oxide red |
| Hiding power on dry coat of 0.04 mm: | 100 % |

<u>Use</u>

*Undercoat.* Paint diluted with water to the ratio, in weight, of 100 / 30; characteristics of the paint after dilution: Specific gravity 1,270 g/l – Solid content by volume 27.6 %; determination of product weight applied on each sample;

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*Second and third coat.* Paint diluted with water to the ratio, in weight, of 100 / 10; characteristics of the paint after dilution: Specific gravity 1.330 g/l – Solid content by volume 34,3 %; determination of product weight applied on each sample and per coat.

The following table shows the consumption and thickness of the paints on the samples examined.

**Table 2. Consumption and thickness of high resin content paints**

**VERSATIC VINYL PAINT CVP 45**

| *Sample N°* | *1* | *2* | *3* | *4* | *5* | *6* | *7* | *8* | *9* | *10* |
|---|---|---|---|---|---|---|---|---|---|---|
| *A – FIRST COAT* | | | | | | | | | | |
| *g / $m^2$ product* | 151 | 156 | 156 | 156 | 166 | 151 | 135 | 140 | 151 | 223 |
| *$cm^3$ product* | 119 | 123 | 123 | 123 | 131 | 119 | 106 | 110 | 119 | 176 |
| *Thickness in m* | 33 | 34 | 34 | 34 | 36 | 33 | 29 | 30 | 33 | 49 |
| *B – TOTAL FINISH* | | | | | | | | | | |
| *g / $m^2$ product* | 275 | 296 | 275 | 280 | 306 | 322 | 291 | 280 | 301 | 296 |
| *$cm^3$ product* | 207 | 222 | 207 | 210 | 230 | 242 | 219 | 210 | 226 | 222 |
| *Thickness in m* | 71 | 76 | 71 | 72 | 79 | 83 | 75 | 72 | 78 | 76 |
| *Total thickness A + B in m* | *104* | *110* | *105* | *106* | *115* | *116* | *104* | *102* | *111* | *125* |

**ACRYLIC PAINT CVP 45**

| *Sample N°* | *1* | *2* | *3* | *4* | *5* | *6* | *7* | *8* | *9* | *10* |
|---|---|---|---|---|---|---|---|---|---|---|
| *A – FIRST COAT* | | | | | | | | | | |
| *g / $m^2$ product* | 177 | 156 | 151 | 145 | 166 | 171 | 161 | 171 | 166 | 182 |
| *$cm^3$ product* | 139 | 123 | 119 | 114 | 131 | 135 | 127 | 135 | 131 | 143 |
| *Thickness in m* | *38* | *34* | *33* | *31* | *36* | *37* | *35* | *37* | *36* | *39* |
| *B – TOTAL FINISH* | | | | | | | | | | |
| *g / $m^2$ product* | 270 | 291 | 296 | 275 | 280 | 280 | 286 | 291 | 312 | 291 |
| *$cm^3$ product* | 203 | 219 | 222 | 207 | 210 | 210 | 215 | 219 | 235 | 219 |
| *Thickness in m* | 70 | 75 | 76 | 71 | 72 | 72 | 74 | 75 | 81 | 75 |
| *Total thickness A + B in m* | *108* | *109* | *109* | *102* | *108* | *109* | *109* | *112* | *117* | *114* |

*Identification of medium resin content paint. UNI 8757*

Type of resin: non plastified acrylic and versatic vinyl paint, both in water dispersion

| | |
|---|---|
| Specific gravity: | 1.460 - 1480 g / litre |
| Solid content by volume: | 37 - 39 % |
| Solid content by mass at 105 °C: | 56 - 57 % |
| Solid content by mass at 450 °C: | 45 - 47 % |
| Calculated C V P : | 59 - 60 % |
| Calculated resin on dry coat at 105 °C: | 17 – 19 % |
| Colour: | oxide red |
| Hiding power on dry coat of 0.04 mm: | 100 % |

<u>Use</u>

*First coat*

Paint diluted with water to the ratio, in weight, of 100 / 30; characteristics of the paint after dilution: Specific gravity 1,325 g/l – Solid content by volume 26,4%; determination of product weight applied on each sample;

*Second and third coat*

Paint diluted with water to the ratio, in weight, of 100 / 10; characteristics of the paint after dilution: Specific gravity 1,410 g/l – Solid content by volume 33%; determination of product weight applied on each sample and per coat.

The following table shows the consumption and thickness of the paints on the samples examined.

**Table 3. Consumption and thickness of medium resin content paints**

**VERSATIC VINYL PAINT CVP 60**

| *Sample N°* | *1* | *2* | *3* | *4* | *5* | *6* | *7* |
|---|---|---|---|---|---|---|---|
| *A - FIRST COAT* | | | | | | | |
| *g / m² product* | 182 | 161 | 171 | 154 | 177 | 151 | 156 |
| *cm³ product* | 137 | 121 | 129 | 116 | 133 | 114 | 118 |
| *Thickness in m* | 36 | 32 | 34 | 30 | 35 | 30 | 31 |
| *B - TOTAL FINISH* | | | | | | | |
| *g / m² product* | 255 | 250 | 281 | 260 | 275 | 312 | 302 |
| *cm³ product* | 181 | 177 | 199 | 184 | 195 | 221 | 214 |
| *Thickness in m* | 60 | 58 | 66 | 61 | 64 | 73 | 71 |
| *Total thickness A + B in m* | *96* | *90* | *100* | *91* | *99* | *103* | *102* |

**ACRYLIC PAINT CVP 60**

| *Sample N°* | *1* | *2* | *3* | *4* | *5* | *6* | *7* |
|---|---|---|---|---|---|---|---|
| *A – FIRST COAT* | | | | | | | |
| *g / m² product* | 161 | 161 | 151 | 135 | 151 | 156 | 151 |
| *cm³ product* | 121 | 121 | 114 | 102 | 114 | 118 | 114 |
| *Thickness in m* | *32* | *32* | *30* | *27* | *30* | *31* | *30* |
| *B – TOTAL FINISH* | | | | | | | |
| *g / m² product* | 281 | 276 | 317 | 270 | 270 | 291 | 291 |
| *cm³ product* | 199 | 196 | 225 | 191 | 191 | 206 | 206 |
| *Thickness in m* | 66 | 65 | 74 | 63 | 63 | 68 | 68 |
| *Total thickness A + B in m* | *98* | *97* | *104* | *90* | *93* | *99* | *98* |

The test samples were then made up in the laboratory. For brickwork conditioning, mortar mixing and laying, and construction of the masonry, the same equipment, methods, procedures were used as normally adopted on the construction site that plans to construct professional masonry.

Before application of the rendering coat and paint and before placing in the artificial accelerated ageing equipment, the test samples were left to set in the environmental laboratory conditions.

## 3 SAMPLE ARTIFICIAL ACCELERATED AGEING EQUIPMENT AND THE AGEING CYCLE

The experimental program, as already mentioned, started in November 1996, which saw the start of collaboration with the Experimental technical laboratory of the Professional university school of Italian Switzerland in Lugano (CH), an experimental laboratory for tests on materials and buildings in construction, already in possession of a climate chamber for accelerated artificial ageing of the test samples, capable of simulating the main meteorological and climatic agents inside the chamber, i.e. rain, sunlight, freezing, dry and humid climates, hot and cold climates.

This equipment offers the possibility of programming any sequence of agents, their duration, and the number of repetitions in the sequence. Some agents may also be activated simultaneously.

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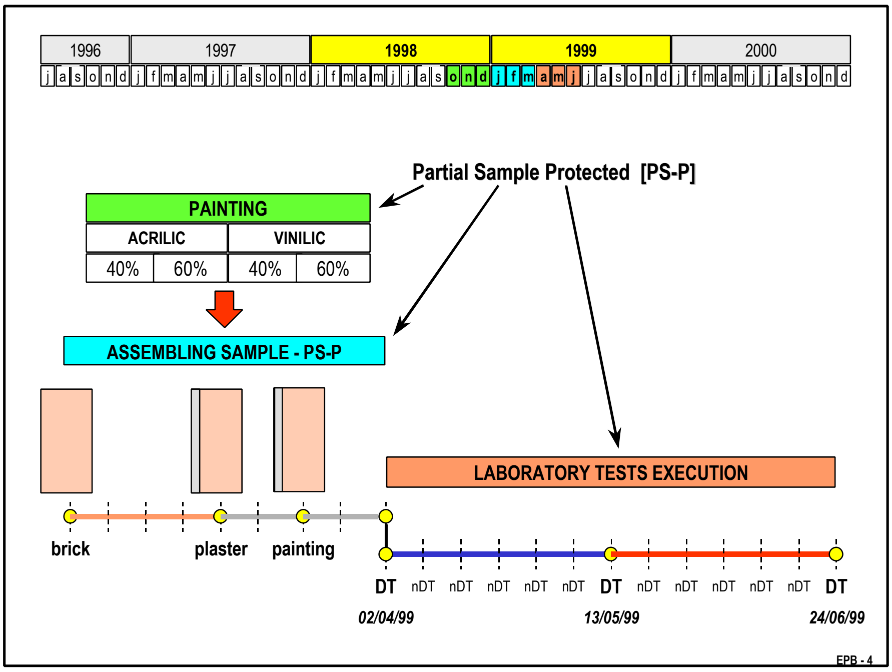


**Figure 3. Program for preparation of partial version-protected samples (October 1998-June 1999)**

The temperature and relative humidity of the air inside the compartment can be regulated respectively from -30 °C to 90 °C and 15% to 95%.

Sunlight is simulated by means of a 6 kW Xenon arc lamp equipped with borosilicate glass filters, the light of which has a spectrum very similar to that of sunlight.

The wetting system comprises four full cone spray points positioned radially at 90° intervals and capable, under a spray angle of 120°, to produce a uniform round coverage spray.

The compartment dimensions are L x D x A = 100 cm x 90 cm x 90 cm, with a capacity to hold four test samples simultaneously.

These are positioned symmetrically around the axis of the lamp as shown in figure 4.

In February 1997, after determining the equipment to be used and the reference laboratory, work moved on to research into standards and bibliography on the possible techniques used for the execution of accelerated ageing tests in laboratories, with the collection of data on standard technical datasheets for the relative interpretation of the various possibilities available. In March 1997 work proceeded to the cataloguing of all possible agents detected in nature, the verification of those that can be simulated in the climate chamber, and the choice of those to be included in the accelerated test cycle.

The decision was to construct an envelope cycle of several agents, to be repeated continuously in the machine, maintaining the possibility of reading the effects of a single agent with the execution of consecutive tests: the effect provoked by the single agent (such as sun radiation) is measured as an effect on an element subject to actions in parallel with other agents, and this is transformed to make the agent more or less effective in terms of deterioration.

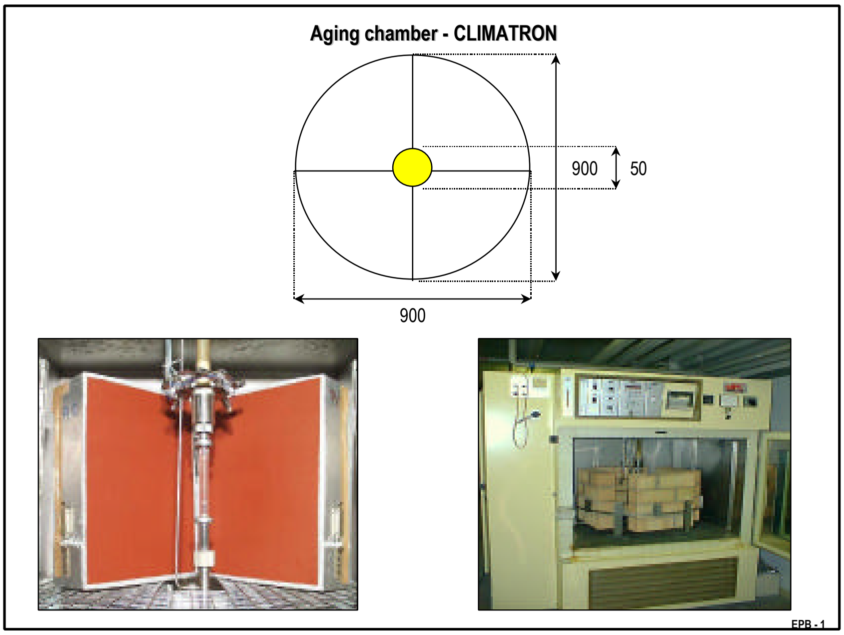


**Figure 4. Climatron climate chamber**

The envelope of the agents in the accelerated test cycle was chosen on the basis of the definition of the maximum intensity that a single agent was to have in the cycle.

This intensity had to be similar to that found in the chosen reference standard of Milan or Lugano, with the subsequent assignment to each sub-phase of an objective to be reached to establish to duration within the cycle.

This objective and the subsequent duration of the sub-cycle were verified through the execution of preliminary calibration tests which enabled the collection of data that was extremely useful to define the artificial accelerated ageing cycle used on the test samples.

As can be seen in figure 5, the cycle has an overall duration of 6 hours and 25 minutes and is structured as follows:

1. Rain phase.

   In this phase, with a duration of 60 minutes, the test samples are sprayed uniformly with water at a temperature of approx. 20 °C.

   Air within the compartment is maintained at a constant temperature of 20 °C and at a relative humidity level greater than 95%.

2. Freezing phase.

   Air within the compartment holding the wet test samples is cooled rapidly to–20 °C and maintained constant for 90 minutes.

3. Hot and humid climate phase.

   During this phase the air temperature and humidity are maintained constant respectively at 55 °C and 95% for 60 minutes. This phase is preceded by a transition period of approx. 80 minutes.

4. Hot dry climate with radiation phase.

   The test samples are irradiated for 80 minutes by means of the xenon lamp. In this interval the climate within the test compartment is kept at a temperature of 30 °C with relative humidity at 40%.

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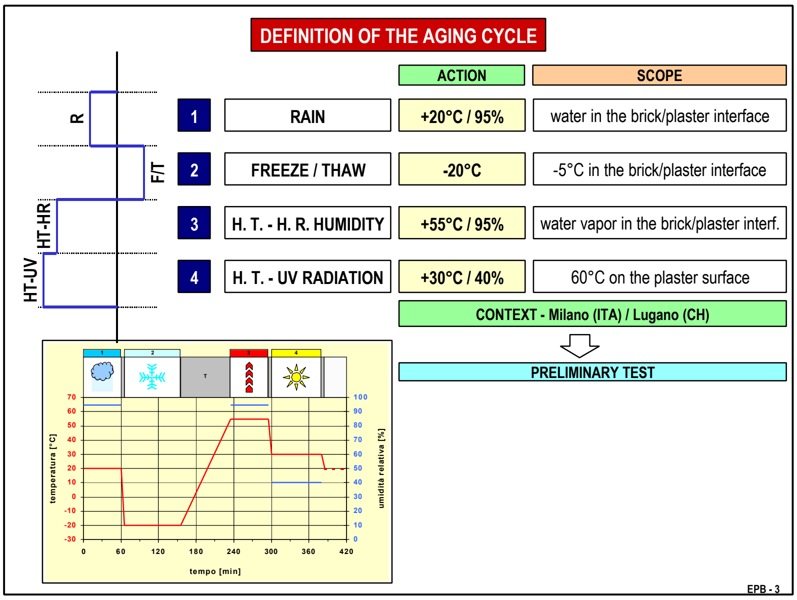


**Figure 5. Artificial accelerated ageing cycle structure**

Before starting the artificial accelerated ageing cycle on the test cycles, in order to enable a study of the physical and performance deterioration of the products under the effect of agents involved in the cycle, further checks were made on individual phases to verify the capacity to reproduce effects from cycle to cycle similar to those produced in natural conditions.

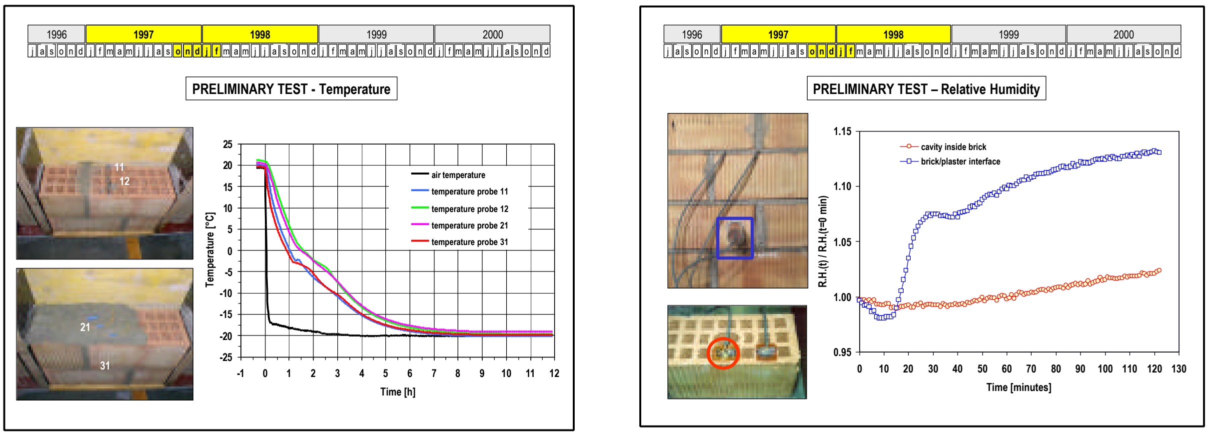


**Figure 6. Preliminary Tests**

For example the temperature was monitored in a test sample over a series of cycles. The data recorded during one of these cycles are illustrated graphically in figure 6.

From the data shown in figure 7 it is clear that the temperature of the rendering at the end of the freezing stage (phase 2) can reach slightly higher than –10 °C (sensor 31) and that the freezing front moves to a depth of a few centimetres behind the rendering-masonry interface (see Curves related to sensors 11 and 22). The trend of the temperature within the test sample at the end of the freezing phase is therefore very similar to the distribution, which, on the basis of the climatic data for the region of Lugano (SIA 381/2) (SIA D 012) and the measurements made on the external brickwork (SIA), would be noted during a cold winter day in the same technical solution exposed to natural climatic conditions. At the end of the hot and humid climate phase (phase 3), the temperature of the rendering and masonry zone immediately behind the interface reaches values from 45 °C to 50 °C. The subsequent radiation phase (phase 4) causes a rise in the rendering temperature to over 60 °C, leaving the wall temperature practically unchanged. Also in phases 3 and 4 the cycle produces, in the test samples, temperatures similar to those produced in natural conditions with and without direct sunlight (SIA).

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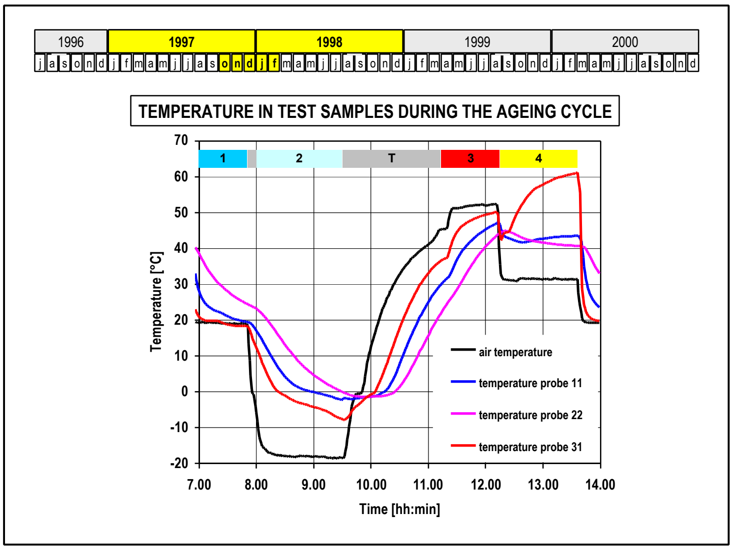


**Figure 7. Temperature in test samples during an ageing cycle**

In conclusion, the data show how the agents involved in the ageing cycle, in the technical solution under examination, produce stress factors comparable to those that the technical solution would be subject to in natural ageing conditions.

## 4 CHARACTERIZATION TESTS

In parallel to the definition of the cycle, the characterization tests were also defined for execution at zero time and during test execution, dividing them into two categories: non destructive, to be performed weekly on the samples, and destructive, to be performed at 150 and 300 ageing cycles. These tests are listed in figure 8. The ageing tests, as mentioned, started with the partial version sample in unprotected and protected types. Figure 3 illustrates the intervals of preparation and ageing of the sample and the agenda for test execution on this sample type. Four samples are placed in the climate chamber directing the outer surface towards the centre of the chamber where the cycle agents originate; on reaching the set ageing interval, the samples are replaced with new ones.

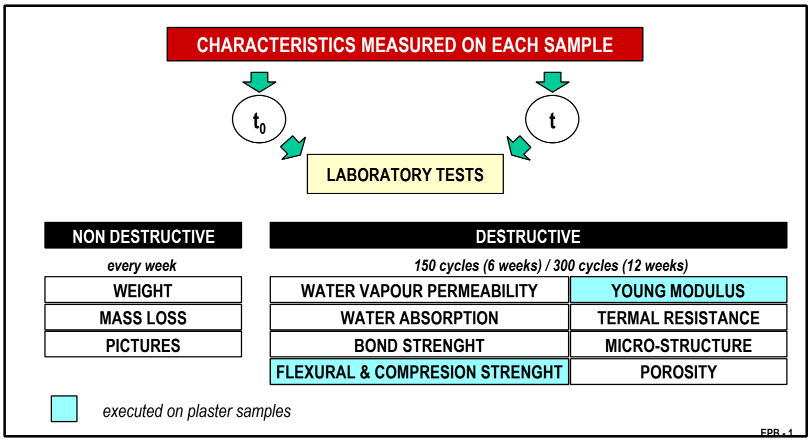


**Figure 8. Characterization test type**

## 5 CONTINUATION OF THE EXPERIMENTAL PROGRAM

The results of the experimental program on the partial versions, performed for the accelerated tests as described above, as well as the relative results for interpretation must be accompanied by the external tests started in May 1999, to enable a reading on a real scale. The choice was made to reconstruct the same elements tested in the accelerated tests according to the type of paint, and to expose them both in Milan and Lugano, environments with similar climatic conditions but with different levels of pollution. In the case of the external tests, two types of exposure were also tested: vertical and at a 45° angle, which, according to standards, should accelerate external ageing by a factor of 6÷8.

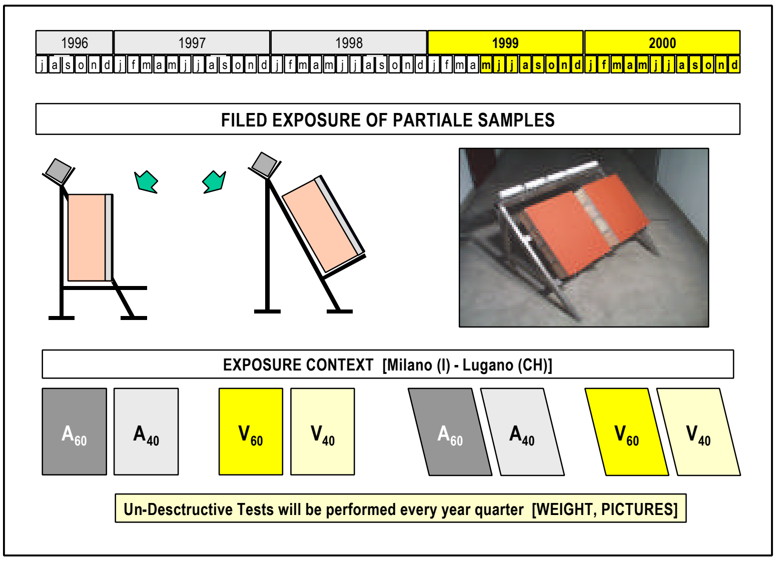


**Figure 9. Field exposure of partial samples**

As this variable does not alter the aspects of deterioration found outside, it enables the interpretation of relative results with those of the accelerated tests much sooner than possible with the vertical inclination tests, even awaiting their final validation. This assumption is also verified on test execution. The other type of sample subject to testing is a sample representing the entire solution: the end of the useful life of the technical solution will be its performance limit, as illustrated in chapter 6, describing the various methods prepared to date by our group for the interpretation and modelling of performance over time. In this case the tests will start in February 2001 and will use the same instrumentation as used previously with a number of modifications (support structure on door for application on the climate chamber).

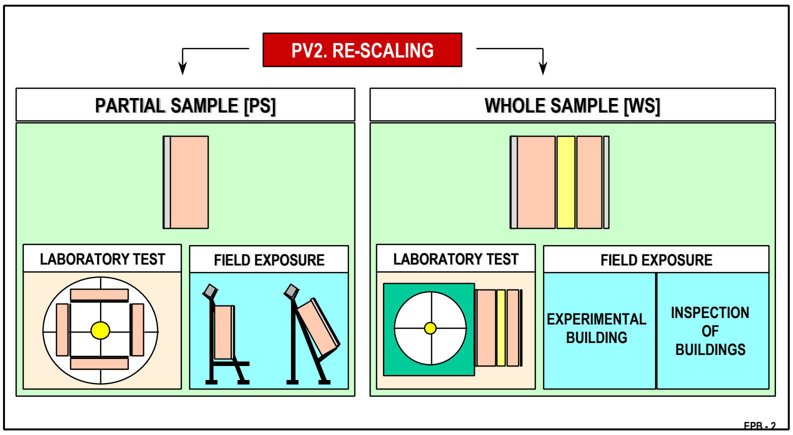


**Figure 10. Overall structure of the experimental program**

The samples in question will reach dimensions four times that of the partial version (surface areas: 100 cm x 950 cm) and will enable a series of significant measurements on a continuous basis (conductivity and humidity at various depths), as well as execution of the characterization tests described above.

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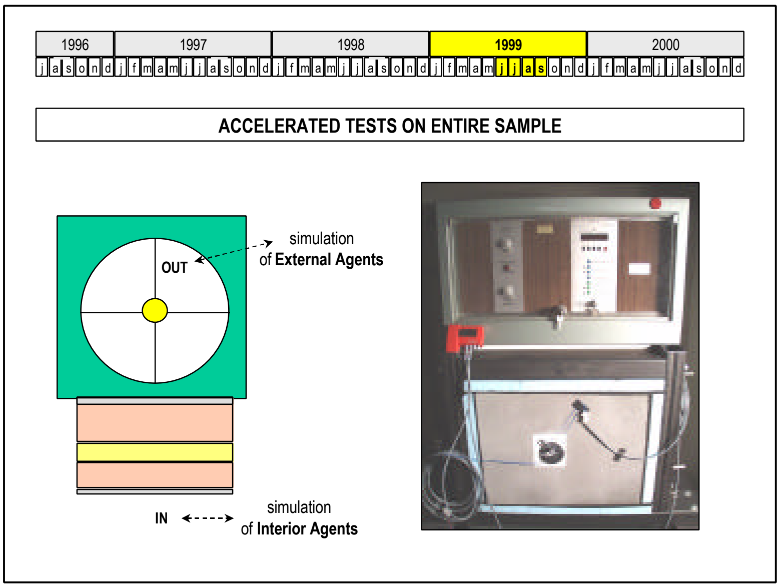


**Figure 11. Characterization test type**

The added value of the sample representing the entire solution lies mainly in the possibility of simulating transitional periods with relative gradients, and thereby verify the effects on deterioration.

This experimental phase will occupy the first 9 months of 2001 and to be completed will have to be compared with external tests in the future to enable rescaling of the results.

The external tests on the entire solution will be carried out in two configurations: by constructing a standard-technological laboratory able to simulate controlled internal conditions in a natural outdoor environment, and through the readings of real new or existing buildings containing the technical solution subject to analysis.

The final representation of the experimental program is therefore the version shown in figure 11, which on completion will be able to be repeated for other solutions in the area or to test alternatives of the same solution with new recipes.

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# Protecting The Building Owner Through Durability Requirements For Building Elements

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**Summary: In order to ensure that tomorrow's buildings will be better (as in the theme of the conference) it has become very important in the construction industry today to predict the life of materials.**
**The New Zealand Building Code clause B2 "Durability" is there to ensure that we have "better buildings for tomorrow" by requiring at the outset that proposed construction components and systems must last a reasonable length of time on a building rather than attempting to punish the person who supplies poor materials when failure occurs years later. The Durability clause has been likened to a fence at the top of a cliff that prevents people from falling instead of an ambulance at the bottom, collecting those who have fallen.**
**This paper will also discuss how the legislation contained in the Limitation Act 1950 and the Building Act 1991 impinges on the durability time limits set out in the building code. The effects of warranties and the Consumer Guarantees Act 1993 are also considered in this context. The Durability clause and the effects of time restraints have to be considered together to ensure that all parties involved in the construction process can interact to ensure that there are better buildings for tomorrow.**

**Keywords: Performance-based building codes, durability, warranties, time limitations**

## 1 INTRODUCTION

### 1.1 The New Zealand Regulatory System

The New Zealand Building Act 1991 introduced the Building Regulations 1992 and a six months lead in period saw the full introduction of the New Zealand Building Code (the building code) as the First Schedule of the regulations in January 1993. This national building code, which is performance-based, was introduced to give consistency to an industry that was previously prescriptively governed by a myriad of Acts, Regulations and local by-laws that riddled the country's building regulatory regime.

### 1.2 The basis for a durability clause

The authors of the New Zealand reforms, the Building Industry Commission, introduced specific requirements for durability in the belief that a building should, with normal maintenance, be required to achieve all performance criteria of the building code, not merely on completion but throughout its intended life. They noted that durability was one of the "user requirements" listed in ISO 6241-1984 (E) *Performance standards in buildings*.

Durability requirements were in place under the previous building control system in two ways:

- By ad-hoc requirements for routine safety inspections (by Government officials) of such building components as lifts and boilers, and annual inspection and licensing (by local councils and the Fire Service) of certain types of buildings (such as apartment buildings and places of assembly).
- By prescriptive requirements for such things as concrete cover to reinforcing and the preservative treatment of timber.
- Under the new system:
- The ad-hoc requirements for routine safety inspections were replaced by a system of routine inspection and maintenance procedures by qualified persons (approved by the territorial authority) for maintenance-sensitive systems such as lifts and sprinklers. Those procedures are specified in a "compliance schedule" for the building issued by the territorial authority. The building owner is responsible for obtaining a compliance schedule and is required to supply the territorial authority with an annual "building warrant of fitness" to state that the procedures have been properly completed.

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- The building code includes performance criteria for the durability of all building elements. Previous prescriptive requirements are generally incorporated as a means of compliance in the Approved Documents.

These regulatory requirements are in addition to any other legal rights to compensation for building failures. These rights are seen as an ambulance at the bottom of the cliff, in contrast to the fence at the top of the cliff provided by the building code.

Although the building code requires a durability of the intended life of the building (indefinite but not less than 50 years) or specific durability periods for some parts of the building, claims for building failures cannot generally be brought to court more than 10 years after the building was erected. That is sometimes seen as an anomaly, but in fact such limitations are common in New Zealand law (and in the law of many other countries), and will rarely entertain a claim for breach of contract more than 6 years after the breach. The true protection provided by clause B2 is not in the right to claim damages but in the integrity of those enforcing the building code. (Benge 2001)

## 2 THE FENCE AT THE TOP OF THE CLIFF

### 2.1 Clause B2 "Durability"

The Objective of most clauses of the building code is to protect the occupier from illness, injury or the loss of amenity with additional objectives for providing access for people with disabilities and some protection of other property. The Objective of clause B2 "Durability" is to:

*"ensure that a building will throughout its life continue to satisfy the other objectives of this code."*

It was generally accepted that elements that were of minor significance and easy to access or replace, such as paint finishes or minor fittings, might need to be replaced after 5 years. Similarly, the claddings in general use at that time were recognised as being sure to last for at least 15 years if properly maintained. No one, however, expects to have to replace the foundations, and of course replacing the building as a whole ends the life of the original building. Thus the Building Act regards buildings generally as having an indefinite life. The exception is made in section 39 of the Building Act for temporary or experimental buildings, referred to in the Act as "buildings having specified intended lives". The owner must specify the intended life at the time of construction, and cannot specify a life of more than 50 years.

Those concepts were used to develop code performance criteria, which included taking into consideration maintenance and the option of specifying a shorter intended life. There are 3 main considerations used to define consumer expectations to determine whether the 5, 15 or 50-year durability requirement applied. These can be put in tabular form as:

**Table 1. Durability Requirement**

| Ease of Access | Ease of Replacement | Detection of Failure | Structural function | **Durability requirement** |
|---|---|---|---|---|
| Difficult<br>OR | Difficult<br>OR | Unable to detect during normal maintenance and normal use<br>OR | Provides structural stability | **The life of the building being not less than 50 years** |
| Moderately difficult<br>OR | Moderately difficult<br>OR | Able to detect during normal maintenance | | **15 years** |
| Easy<br>AND | Easy<br>AND | Able to detect during normal use | | **5 years** |

All conditions must exist in order to be able have a 5 year durability. If a material does not fit into that category it falls into the worst of the other two categories, ie if the worst condition is one of those in the first line above then the building element must have an indefinite durability (the life of the building being not less than 50 years), if the worst condition is one of those in the second line above, then it will only require a 15 year durability.

Thus the Performance criteria are:

#### 2.1.1 B2.3.1

*Building elements must, with only normal maintenance, continue to satisfy the performance requirements of this code for the lesser of the specified intended life of the building if stated, or:*

*a) The life of the building, being not less than 50 years, if:*

i) *Those building elements (including floors, walls and fixings) provide structural stability to the building, or*

ii) *Those building elements are difficult to access or replace, or*

iii) *Failure of those building elements to comply with the building code would go undetected during both normal use and maintenance of the building.*

b) *15 years if:*

i) *Those building elements (including the building envelope, exposed plumbing in the subfloor space, and in-built chimneys and flues) are moderately difficult to access or replace, or*

ii) *Failure of those building elements to comply with the building code would go undetected during normal use of the building, but would be easily detected during normal maintenance.*

c) *5 years if:*

i) *The building elements (including services, linings, renewable protective coatings, and fixtures) are easy to access and replace, and*

ii) *Failure of those building elements to comply with the building code would be easily detected during normal use of the building.*

### 2.2 Approval at building consent stage

Under section 34 of the Building Act

*The territorial authority shall grant the consent if it is satisfied on reasonable grounds that the provisions of the building code would be met if the building work was properly completed in accordance with the plans and specifications submitted with the application.*

There is little specific guidance given to the territorial authorities except in section 47 of the Building Act which states that

*In the exercise of its powers under sections 30 to 46 and 64 to 71 of, and the Third Schedule to, this Act the territorial authority shall have due regard to the following matters:*

*(a) The size of the building; and*

*(b) The complexity of the building; and*

*(c) The location of the building in relation to other buildings, public places, and natural hazards; and*

*(d) The intended life of the building; and*

*(e) How often people visit the building; and*

*(f) How many people spend time in or in the vicinity of the building; and*

*(g) The intended use of the building, including any special traditional and cultural aspects of the intended use; and*

*(h) The expected useful life of the building and any prolongation of that life; and*

*(i) The reasonable practicality of any work concerned; and*

*(j) In the case of an existing building, any special historical or cultural value of that building; and*

*(k) Any other matter that the territorial authority considers to be relevant.*

When considering those aspects above, a territorial authority can obtain information about the durability of a building elements from:

- Appraisals
- History in use
- Reputation of the manufacturer
- Manufacturer’s warranty.

The first three are important considerations, and although the manufacturer’s warranty is significant, it should not be the sole criterion for an assessment of durability. A warranty of X number of years is worth nothing if the company offering it goes into liquidation before that expiry of X years. Warranties for up to 25 years are sometimes offered by larger firms that have a stable history of existence but it is hard to envisage a warranty for 50 years or the life of the building being taken seriously.

If the warranty is only for 5 years, it does not detract from the fact that the building material or component may have a realistic life of 15 or 20 years. An analogy to this would be a new car that comes with a 5 year warranty. No one expects it to grind to a halt at the end of 5 years. With normal maintenance and the odd part renewed, a new car could be reasonably expected to last 20 years. Some territorial authorities recognise this fact and will accept a statement (known as producer statement for the purposes of a building consent) from a person recognised as expert in the field about the durability of a building element.

### 2.3 Issuing of code compliance certificate

When a building is considered to be completed the owner must apply to the territorial authority for a Code Compliance Certificate (CCC). The date of issue of a CCC is important because that is when the clock starts for the durability period. An interim CCC can be issued when there is some doubt about compliance of specific elements so that the commencement of the durability period of most of the building elements is not held up by minor matters.

The territorial authority normally issues the CCC after a final inspection or after receiving a building certificate from a building certifier. Withholding the CCC is normally sufficient to ensure proper completion of the work but under section 42 of the Building Act the territorial authority may issue a notice to rectify if necessary. To give weight to this provision the Building Act under Section 80 allows for a fine not exceeding $200,000 with $20,000 per day for not rectifying. No fines to this extent have ever been imposed however.

The CCC can act as a backstop if something has been missed at the time of the issuing of the building consent, or can pick up inappropriate substitution of building elements.

## 3 THE AMBULANCE AT THE BOTTOM OF THE CLIFF

### 3.1 General Limitations

Despite the protection that the durability clauses of the building code gives to a building owner, other factors can reduce their effectiveness. In particular, the Limitation Act 1950 and section 91 of the Building Act can affect the time spans set down in the B2 clauses.

For the purposes of this paper, building owners have been divided into the two categories of “initial owner” and “subsequent owner”. An initial owner is the person who has the contract with the builder or developer while a subsequent owner is the person who has purchased a building from a previous owner. These distinctions are important as the time limitations vary in each case. It is also emphasised that the main contractor will invariably be responsible to an initial owner for the negligent acts of subcontractors.

In simplified terms, the Limitation Act says that claims must be brought before a Court or a tribunal within a certain time, which is called “the limitation period”. These basically say that;

- For claims in contract, the limitation period is 6 years from the time that any defective work was carried out.[1]
- For claims in tort, the limitation period is 6 years from the time of *discovering* a defect, or from when it ought to have been discovered by a careful inspection.

Section 91 of the Building Act introduces a 10-year longstop for claims arising out of the construction, alteration, etc of buildings. This provision makes no difference to most claims for breach of contract, where the 6-year limitation will apply. It does, however affect a claim in tort, a tort being a civil wrong that is not a breach of contract. For example, under the Limitation Act, even if a defect is first discovered 20 years after a building was constructed, an owner would have a further 6 years to make a claim in tort. However, the Building Act sets a time limit of 10 years from when the negligent work *was carried out* to make such a claim.

As an initial owner has to sue under the contract, he or she cannot take an action against a contractor for negligent work once the 6-year time span has been reached, even though the building code requires a 15-year durability requirement for a specific item.

On the other hand, as a subsequent owner is not able to sue a negligent builder in contract, he or she must claim against a contractor in tort if it can be shown that any defective work was due to the builder’s negligence. This means that a subsequent owner may have a maximum 4-year limitation time advantage compared with an initial owner. (The subsequent owner could be the first resident of the building, having bought it from a developer.) For example, if the faulty work was carried out 7 years previously and the fault is just discovered, the owner would have a further 3 years to commence an action in tort. Although there is some uncertainty as regards the present attitude of the New Zealand Courts to concurrent liability (the right to sue in tort where one already has a contract), it seems likely that an original owner would not be able to sue in tort to take advantage of this differing approach.

[1] This refers to a simple contract into which category most domestic contracts would fall. In the case of a deed, the limitation is 12 years.

Building defects generally become evident during construction or within the next few years but some defects, particularly durability defects, could take many years to appear. Nonetheless, once the limitation period has expired, no action can be taken against a negligent contractor even though the durability or other requirements have not been met.

### 3.2 Warranties

Some additional protection for building owners is available if they are provided with a warranty or guarantee for building elements requiring durability under the building code. A warranty or guarantee is a commercial arrangement between a manufacturer and a buyer. These can be made out in general terms, which usually apply only to the original owner, but they can be specific to both subsequent and original owners. However, as mentioned previously, a warranty or guarantee is worthless if the organisation offering it goes into liquidation.

In *Richards Bros Ltd v Page* (Holderness 1994) it was said that:

> It is not uncommon for a breach of warranty to arise from events long past and the general law as to limitation (as provided for in the Limitation Act 1950) does not bar proceedings based upon a breach simply because the events rather than the breach of the warranty occurred more than six years before the commencement of the proceedings.

In accordance with this judgement, the limitation clock starts when the warranty or guarantee is invoked, which would normally be the time that a defect was discovered. So, for example, if an original building owner had a 15-year durability warranty, the limitation deadline could be extended from 6 years to a total of 15 years.

Both of the major New Zealand Standard Construction Conditions of Contract refer to the provision of guarantees. Clause 11.5 of NZS 3910:1996 Conditions of Contract for Building and Civil Engineering Construction states:

> 11.5.1 The Contractor shall provide the Principal with written guarantees where required by the Special Conditions.
>
> 11.5.2 Such guarantees shall be supplied to the Engineer in writing before the Engineer issues the Defects Liability Certificate and shall be in the form required by the Special Conditions.

In the First Schedule “Special Conditions of Contract”, there are the options to either list the guarantees that are required or to indicate that no guarantees are required.

Rule 53 of the NZIA Standard Conditions of Contract SCC1 2000 states:

> The Contractor must provide the Principal with the specified guarantees.
>
> 53.1 The wording must comply with any requirements in the Specifications.

Addendum B1, “The Specific Conditions of Contract” requires guarantees from the Contractor before the Principal can use the Contract Works.

These standard clauses allow for the inclusion of properly worded guarantees or warranties within building contracts that can ensure that at least the 15-year durability times can be met. In order to reinforce the durability requirement of the NZBC such guarantees or warranties should be so worded that they are able to be passed on to subsequent owners.

### 3.3 The Consumer Guarantees Act 1993

The Consumer Guarantees Act 1993 (the CGA) gives additional protection to those who are “consumers” under the Act. Its clauses are implied in any contract involving a consumer, and in relation to the building industry it will generally apply to the construction of domestic dwellings and the like. A consumer is defined under section 2(a) of the Act as:

> A person who acquires from a supplier goods or services of a kind ordinarily acquired for personal, domestic, or household use.

Whereas the CGA is still subject to the Limitation Act, it will impact on the rights of original owners. Section 55 of the Act amends section 63 of the Building Act, which now reads that

> Nothing in this Part of the Act shall derogate from the provisions of the Fair Trading Act 1986 or the Consumer Guarantees Act 1993.

In particular, the Act sets out various implied guarantees for both goods and services that the supplier of these cannot contract out from except for business transactions. In the case of a building, services would include the work required to put the materials (goods) in place. The CGA does not apply to goods that are supplied by competitive tender, nor does it give any protection to subsequent owners.

The CGA provides a list of remedies available to consumers where goods or services do not comply with guarantees. Such remedies might circumvent contractual limitation or exclusion clauses that are detrimental to an original owner.

Where services do not comply with the guarantees set out in the Act, a building owner may also obtain damages for foreseeable loss or damage to other elements that are affected by the failure of a specific element.

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The CGA gives a consumer a right of redress against a manufacturer who has given any express guarantee that is binding on the manufacturer in accordance with the Act. This remedy could be invoked where, for example, the contractor has become insolvent.

It has also been suggested that as a consumer "acquires" goods as defined in the Act, then this may give a consumer a right to directly sue an individual subcontractor rather than just the builder who is a party to the contract (Fraser1994).

## 4 CONCLUSION

This brief discussion on the limitation placed on durability time-scales indicates that it may be difficult to take action to get defective work rectified. In some instances a negligent contractor will escape any legal liability once 6 years has elapsed from the time any faulty work was carried out. While the Consumers Guarantees Act may offer additional remedies to building owners, it is still encapsulated in the Limitation Act requirements. Only when an effective warranty has been given can some owners enforce the 15-year durability clause of the building code. The 50-year requirement is, of course, entirely out of reach.

It seems anomalous that the building code should specify durabilities exceeding the long-stop limitation period of 10 years, because any durability inadequacy that is discovered after that time cannot be the subject of a claim before a Court or tribunal. However, the building code is not intended to create rights between parties, it is intended to be enforced by territorial authorities. The situation is that the territorial authority is required to refuse building consent if it is not satisfied on reasonable grounds that all proposed building elements would achieve the durabilities required by the building code. Further more, at completion of the building, the territorial authority cannot issue a code compliance certificate unless the building complies with the code, and can issue a notice to rectify if it does not comply.

As the "ambulance at the bottom of the cliff" is not fully equipped to deal with durability problems in the building industry, it rests with local authorities and independent building certifiers in New Zealand to ensure that buildings are built in accordance with the building code.

The adage "Do it once and do it right' is of paramount importance.

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# A Performance Assessment of Flood-Damaged Shearwalls

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**Summary: Flood events affect more people than any other form of natural disaster. Flooded structures are partially submerged and then re-dried, which leads to swelling and shrinking over an arbitrary wall height. One repair scheme is to remove the interior cladding (drywall), dry the wood system, and re-install new drywall. This investigation was conducted to evaluate the effects of long-term water exposure on the mechanical properties of oriented strand board (OSB) sheathing and the impact of those changes on the capacity of light-frame shearwalls. OSB was submerged for up to seven days, air-dried and then tested for embedment strength, shear strength through the thickness, and shear modulus. Nine walls were built for the test program. Three were tested at ambient conditions and monotonically loaded in shear to establish the displacement criteria for the quasi-static methods; three walls were tested at ambient conditions (control walls) using a quasi-static loading protocol; and three were submerged in 1 m of water for seven days, re-dried, and then tested using the same quasi-static protocol as the control walls. The results from the OSB material tests showed that most of the material property degradation occurred in the first 48 hr of submersion, for example, the embedment strength was reduced by almost 40 percent after 48 hr and showed no further reduction even with 120 hr of additional soaking. The shearwall test results did not follow the materials' results -- the quasi-static shearwall tests showed that water submersion did not reduce shearwall capacity, the energy absorption of the wall, or change the yield mode. However, shearwall stiffness was reduced, apparently the result of reduced embedment stiffness of the OSB sheathing.**

**Keywords: Shearwall; monotonic testing; quasi-static testing; timber; durability**

## 1 INTRODUCTION

Durability of structures has been the focus of study and design innovation in recent years. The design engineer implements design features and materials that, when used with appropriate construction methods, yields buildings that have long-term durability (TenWolde and Rose 1994). The presumptive design assumption is that the condition of the building system will remain unchanged throughout its service life. However, after a flooding event, there may be a loss in capacity to resist major loading events. Designing for durability requires quantification of the changes in the mechanical properties and resulting performance of the structure. Condition assessment combined with quantified data on the impact of the given material and connection condition provides a vital link to predicting residual structural capacity to resist future loading events given the existing level of damage that is revealed by physical inspection. At present, durability assessment has incorporated only the observed condition of the materials and essentially a go/no-go" basis for replacement. It would be desirable to include the existing levels of damage in the framing and sheathing components in the fragility of residual life assessment of a structure after a natural disaster.

Flooding is a natural disaster that affects more people world wide than any other form of natural disaster (Hausmann & Perils 1998). Geo-scientists have reported that floods account for 23 percent of global natural catastrophes but accounted for 67 percent of casualties and caused 53 percent of the economic losses in the US (Munich Re 2001). The insurance industry has many definitions of "flooding" because flooding can originate from many different events and takes many forms. In general, the insurance industry agrees that a flood is a general and temporary inundation of normally dry land from the overflow of inland or tidal water, rapid accumulation of runoff of surface waters, mud flows or mud slides, or waters due to the collapse of shores and retaining systems. Data reported by Hausmann and Perils (1998) showed that most flood events in buildings involve water depths less than 2 m. The number of affected properties can be significant, as in 1993, when approximately100,000 houses were damaged by flooding in the Mississippi River Basin. Thus, flooding represents a potential hazard to the durability and serviceability of residential and industrial buildings.

### 1.1 Technical Background

Flooding in residential structures has the potential to affect the sheathing, the wood frame and the nailed connection. The literature has many reports of accelerated aging tests and durability studies of wood-based panel products. The various accelerated aging protocols are generally correlated to a given number of years in service under a stated exposure condition. One of the most important factors causing deterioration or strength reduction of wood-based panels is the change in moisture content (Suchsland 1982). Changing moisture content leads to permanent thickness swelling that consists of two parts: swelling of the wood and release of compression stresses that were induced into the panel at the manufacturing process (McNatt 1982). The standard methods of test for water absorption and thickness swelling and moisture-related effects on wood-based panel materials are based on 24 and 48 hr tests. However, flood events do not correlate to accelerated aging protocols, and flood events can last much longer than the test protocol durations. Hence, a method of testing the effects of water exposure for longer periods than 48 hr may be needed.

Shearwalls and nail connections between sheathing and the wood framing have been studied extensively. Neisel & Guerrera (1956) showed that racking strength and lateral nail resistance vary exponentially with moisture content and that racking strength is correlated to lateral nail resistance. Toumi & McCutcheon (1978) used energy method to develop a method of predicting racking strength of light-frame walls. Patton-Mallory & McCutcheon (1987) predicted racking displacement-load behavior of a shearwall from the known behavior of the fasteners and sheathings that gave accurate predictions for small-scale walls. Their models showed that wall displacement was the sum of fastener deformation and displacement due to shear deformation in the sheathing. Shear deformation in the sheathing contributed only 10 to 15 percent of the wall displacement. Hence, most of the deformation in the shearwalls can be attributed to the nail deformation. Others have written more sophisticated models for nonlinear dynamic analysis; the most recent computer model incorporates shear modulus, nail hysteresis, and shear strength to predict load-displacement characteristics and energy dissipation under general cyclic loading (Folz & Filiarault 2001). The computer models and earlier closed-form models can assess the performance of shearwalls with nail and stiffness properties that are constant over the shearwall. The effect of a flood exposure over some arbitrary height of the wall would have to be approximated by evaluating the limits, which could be a wall that has not been exposed to water and a wall that was fully submerged.

Nailed connection capacity and yield mode depend on the embedment strength of the main and side members, bending strength of the nails, and geometry of the joint (NDS 1997). The yield mode of a nailed connection can be managed by using different materials or by changing the geometry. Chou & Polensek (1987) studied damping and stiffness of nailed joints as they were affected by the drying of the green lumber using cyclic load tests of single nailed joints. Damping ratios and slip moduli were significantly smaller for joints with gaps than for those without gaps. Variations of interlayer surface roughness did not appear to significantly affect either damping or stiffness. Mohammad & Smith (1996) reported the results of a study with OSB-lumber nailed connections that were subjected to various moisture cycling regimes between 5 and 15 percent moisture for samples made with dry lumber and between 5 and 12 percent for samples made with green lumber. The mechanical testing used a cyclic test protocol and showed that the moisture cycling had an adverse effect on connection stiffness. Then, if a structure was flooded, re-dried, and a gap developed between the stud and the sheathing as a result of shrinkage, the stiffness and damping of the building would be adversely affected.

The Consortium of Universities for Research in Earthquake Engineering (CUREE) grew from the observation that woodframe structures did not fair well in some recent seismic events. They have sponsored many studies on shearwalls for the purpose of improving hazard mitigation. Researchers expressed a need for full-scale tests of on wood structures that use a standard design in order to make results comparable (Zacher 1999). For this reason, a quasi-static test method was developed by CUREE to assess the lateral force resistance and energy–related properties of shearwalls. The method was adapted for this study.

### 1.2 Objectives

The duration of water events often exceeds 48 hr, which is the period used to characterize moisture effects on wood-based panels. Then, an important structural issue exists: Does long-term water exposure cause a reduction in capacity of shearwall systems? It is logical to presume that significant changes in wall performance would result for structures that have experienced one or more water events where the sheathing has been wet.

The specific objectives of the study were:

- To investigate the effect of long-term water exposure on the material properties of the shearwall constituents;
- To investigate the effect of long-term water exposure on static and energy-related properties of shearwall assemblies.

## 2 MATERIALS AND METHODS

### 2.1 Wood-Based Materials

Twenty-seven panels of OSB were purchased from a local lumber yard. The OSB was APA rated sheathing and rated for EXPOSURE 1 (APA 2000), which means that the adhesive was considered to be fully waterproof. The panels were 11.9 mm (15/32 in.) thick and 1220 by 2440 mm in plane dimension. The lumber for the study was No.2 Douglas fir-Larch, 38 by 89 mm, which by visual inspection was determined to be Douglas-fir.

### 2.2 Materials Properties Tests

Five OSB panels were randomly selected for the materials properties tests and the remaining panels were used for wall construction. The five panels were cut in half resulting in ten half-sheets, 1220 mm in length and width.

The engineering properties to be tested were embedment strength, edgewise shear strength, and shear modulus. These were to be tested following the standard methods given in ASTM (2001a, 2001b, 2001c). The embedment strength of the OSB was assessed in only the principal direction of the panel because it has been shown that embedment strength of small diameter fasteners is not affected by angle to the grain. The diameter of the nail used for the embedment tests was 2.9 mm. The edgewise shear strength was tested in both principal directions of the panels. Shear modulus was tested using square plate. The test was conducted on both diagonals of the plate and the result was the average of the two tests on each plate. The specimens were located on the replicate panels as shown in Fig 1. Each test and water exposure period was assigned to each half-sheet of OSB, so in the end, n = 10 for each of the exposure periods for each test.

The treatments to be applied were various periods of water exposure: 0, 24, 48, 96, and 168 hr of submersion in fresh water. The specimens were soaked in a single batch and designated specimens were removed from the water at the specified times. The specimens were air-dried after the water exposure.

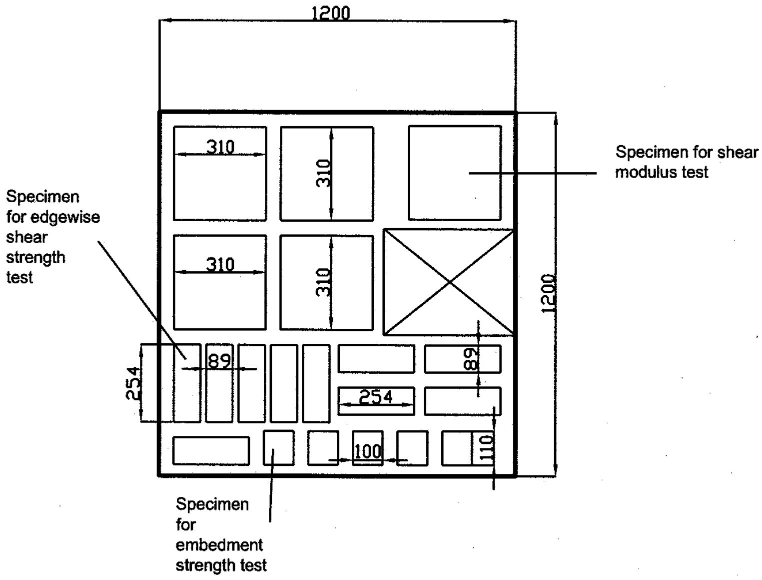


**Figure 1. Cutting diagram for materials properties testing.**

### 2.3 Shearwall Preparation

Nine shearwalls, 2440 by 2440 mm, were built like those by Krawinkler *et al.* (2000) with the exception of nail spacing, which matched the nail spacing of a recent study by Langlois (in preparation). The 38 by 89 mm studs were placed at 406 mm centers. The OSB sheathing was oriented vertically on the stud frame and pneumatically nailed with 8d box nails (60.3 by 2.9 mm). The OSB panels were installed with a 3-mm gap between them. The end studs were doubled and nailed together with 16d box nails (82.6 by 3.3 mm). The top plate was a double member while the sill plate was a single member. Tie-down straps were nailed to the end studs and bolted to the foundation 109 mm from the ends of the sill plates. Two other anchor bolts were located at 610 mm from each end of the sill plate.

Three of the shearwalls were partially submerged in fresh water by placing them in a specially constructed tank so that the sill plate and the lower 1 m of the wall were below the water line. These were kept in the water tank for 168 hr (seven days). At the end of the water-exposure period, the walls were lifted out of the tanks and stood upright in a covered outdoor area to dry. A handheld resistance-type moisture meter was used to check the moisture content of the studs during the drying period. One wall was used as an indicator wall, that is, it was weighed several times during the drying period and used to indicate the dryness of all three walls.

### 2.4 Shearwall Tests

**Monotonic Tests**. Three shearwalls were subjected to monotonic testing, which is a ramp load to failure in one direction. The racking load was applied to the top of the wall through a steel C-channel that was bolted to the top plate of the wall. The wall was attached to a foundation fixture with four anchor bolts. Four LVDTs for displacement measurements were positioned as shown in Fig 2. The LVDTs had an error of less than 0.1 percent. Force was measured by an electronic load cell mounted on the end of the actuator. The displacement was applied at a uniform rate of 0.254 mm/min. Load and displacement at the actuator and the LVDTs were recorded at 10 Hz. Control and data acquisition were implemented via Labview® software.

The properties to be determined from the monotonic shearwall tests were stiffness (N/mm), racking strength (kN), energy absorption (kN*mm), and the reference displacement (mm). The stiffness was calculated as the secant modulus between the origin and a point 0.8 of the maximum load ($F_{max}$) in the ascending portion of the displacement–load curve. The racking strength was taken to be $F_{max}$. Energy absorption was calculated as the area under the displacement-load curve between the origin and 0.8 $F_{max}$ on the degrading slope of the displacement-load diagram. The point 0.8$F_{max}$ on the degrading slope was defined as "failure" (Ceccotti 1999). The reference displacement was calculated as 0.6 times the displacement at failure.

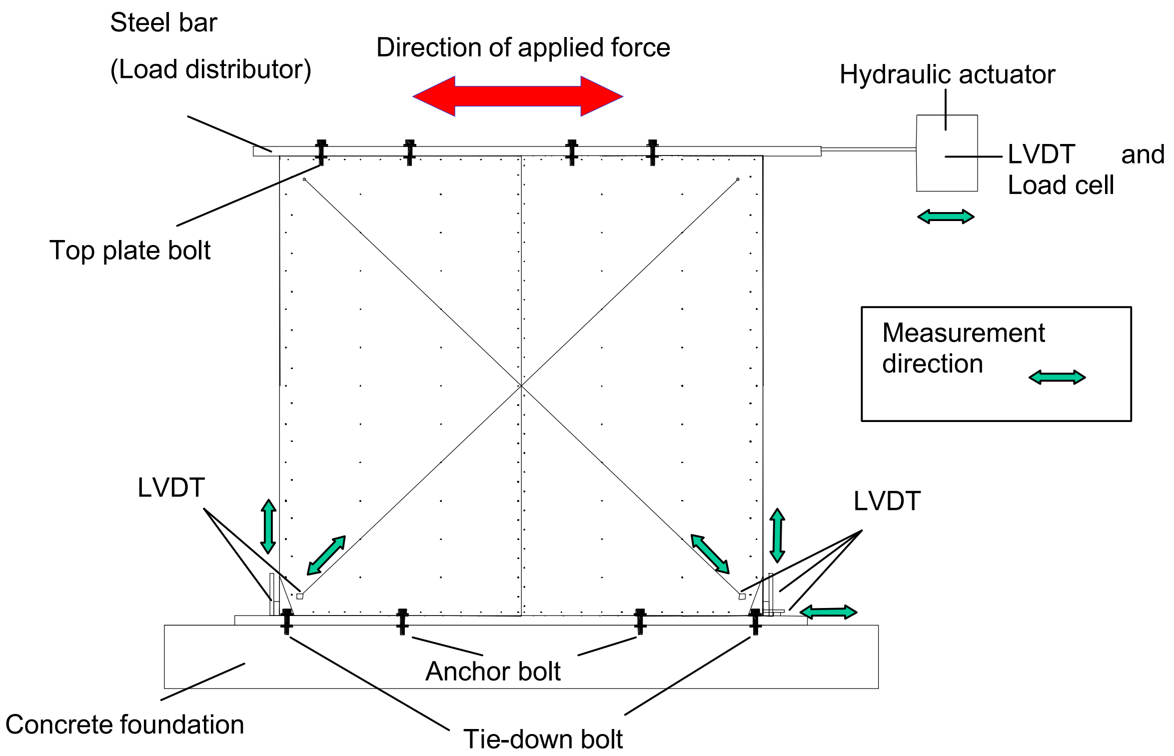


**Figure 2. The shearwall test apparatus.**

**Quasi-Static Tests.** The quasi-static tests were a fully reversed-cyclic test that followed the protocol for ordinary ground motions (Krawinkler et al. 2000). The apparatus and method of load application was the same as for the monotonic tests. Data were recorded at 25 Hz. The time-displacement function was based on the reference displacement of the monotonic test. The test protocol began with a series of small amplitude initiating cycles. Then, each amplitude change was led by a single leading cycle followed by two or three trailing cycles at 0.75 of the current leading cycle amplitude. The time-displacement diagram for the quasi-static tests is given in Fig 3.

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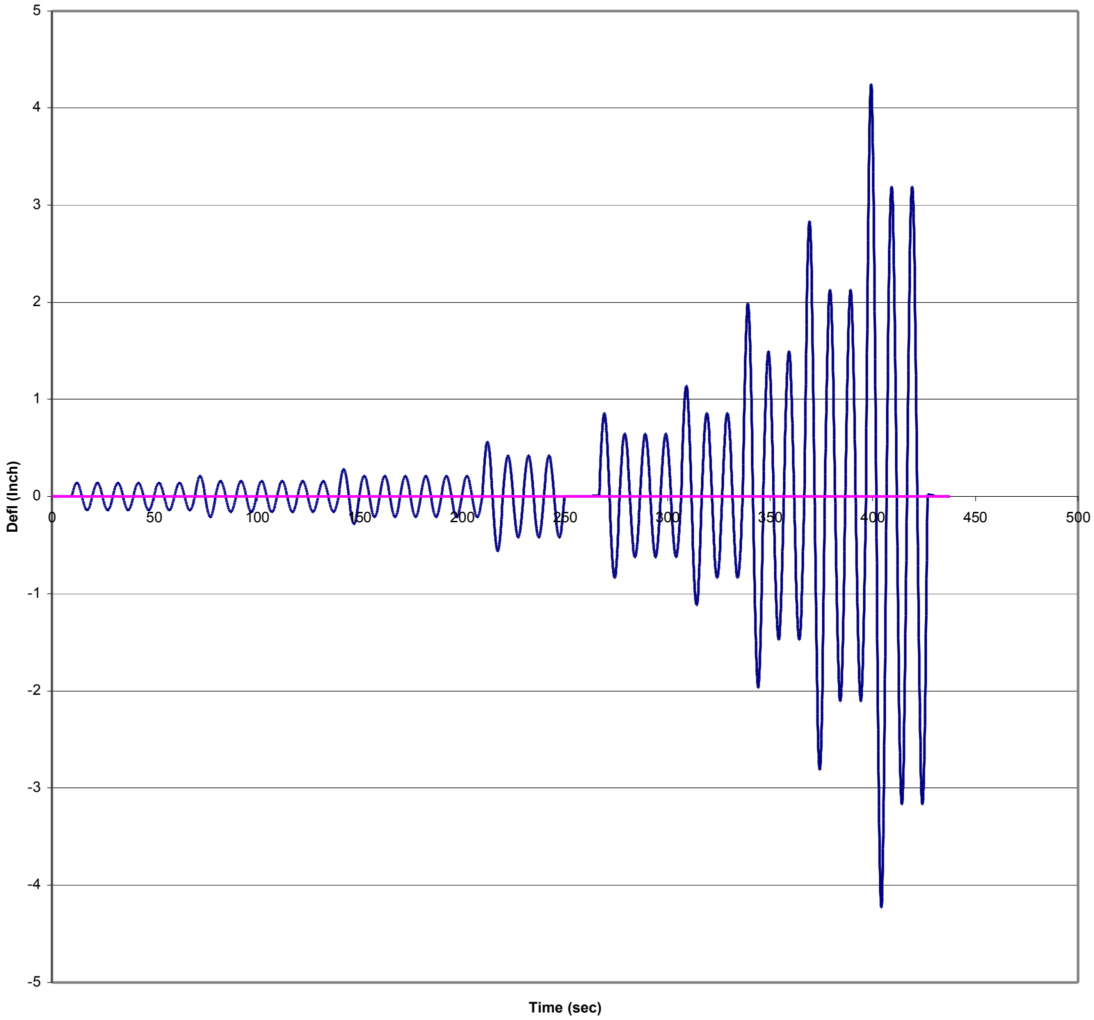



**Figure 3. Time-displacement diagram for the quasi-static shearwall tests.**

The properties to be assessed by using the quasi-static shearwall test data were stiffness (N/mm), backbone energy absorption (kNAmm), total energy absorption (kNAmm) and racking strength (kN). The initial stiffness was calculated as the secant modulus of the cycle that included the maximum load. The line was drawn from the origin to a point at 0.8 of the maximum load in the ascending portion of the designated cycle. The backbone energy absorption was calculated as the area under the backbone curve extending from the origin to the displacement at $0.8F_{max}$ on the degrading slope of the backbone load-displacement curve. The total energy absorption was calculated as the sum of the areas in all cycles up to the first post-peak primary cycle where the highest value was less than or equal to $0.8F_{max}$. Finally, the racking strength was the maximum load measured during any cycle of the test.

## 3 RESULTS AND DISCUSSION

### 3.1 Materials Properties

The results of tests with OSB that was submerged in water from 0 to 168 hr are summarized in Table 1. The embedment strength, edgewise shear strength, shear modulus, and specific gravity follow the same trend – most of the loss in properties occurred in the first 48 hr of water soaking. After that time, the losses were minimal. Water absorption at 48 hr was 43 percent, but by 186 hr water absorption had reached 73 percent. The residual thickness swelling, that is the thickness swelling after being re-dried, remained at the same level for soaking periods of 48 hr and greater. It is noted that the moisture content after re-drying did not return to the initial moisture content. In fact, the moisture content in the desorption process returned to only 11.5 percent, which was 4 percent greater than the initial condition. Some of the properties loss is attributed to the somewhat elevated moisture content. Embedment stiffness, the slope of the linear portion of the embedment-force diagram, shifted from 95 N/mm to 60 N/mm after 24 hr of water soaking and did not change with further soaking. To summarize the results of the materials investigation, it appears that embedment strength and shear modulus are most severely affected by the long-term water exposure .

**Table 1. Summary of test results from OSB water exposure tests when the OSB was tested after re-drying. Parenthetical values are standard deviations.**

| Property | Water exposure period (hr) | | | | |
|---|---|---|---|---|---|
| | 0 | 24 | 48 | 96 | 168 |
| Embedment strength (MPa) | 35.3 (13.06) | 29.1 (4.92) | 21.2 (6.66) | 21.4 (7.62) | 25.0 (6.25) |
| Shear strength – parallel (MPa) | 11.1 (0.89) | ----[a] | 9.7 (1.13) | 7.9 (0.25) | 8.6 (0.73) |
| Shear strength – perpendicular (MPa) | 10.6 (1.73) | 8.8 (0.53) | 8.1 (0.61) | ----[a] | 7.7 (0.76) |
| Shear modulus (GPa) | 1.8 (0.35) | 1.3 (0.15) | 1.2 (0.17) | 1.2 (0.16) | 1.1 (0.15) |
| Moisture content[b] (%) | 7.6 (0.42) | 9.9 (0.36) | 10.9 (0.41) | 11.4 (0.67) | 11.5 (1.24) |
| Specific gravity | 0.62 | 0.59 (0.02) | 0.55 (0.02) | 0.53 (0.02) | 0.55 (0.04) |
| Water absorption (%) | | 22.2 (1.22) | 43.2 (4.54) | 61.8 (4.39) | 73.1 (9.14) |
| Thickness swell (%) | | 6.5 (1.31) | 12.1 (2.47) | 15.0 (2.88) | 14.1 (3.37) |

[a] problems with test fixture, data censored from data set.

[b] moisture content of the embedment specimens when tested.

### 3.2 Shearwall Tests

The monotonic and quasi-static tests were conducted over a two-week period. The mean monotonic test results (n = 3) showed that the capacity of the walls was 29.1 kN, the energy absorption was 2759 kN*mm, initial stiffness was 870 N/mm, and the reference displacement was 71 mm. The monotonic test data were overlaid and are shown in Fig 4.

After the submersion period, the shearwalls were weighed and reweighed again prior to testing. On the average, each wall absorbed 22.7 kg of water. The walls were considered to be air-dry when the weight of each wall returned to within 3 kg of the initial weight, which took about 14 days. Moisture content of the studs, as determined by using a resistance moisture meter, was found to be approximately 19 percent at the time of testing. Materials tests suggest that the moisture content of the OSB sheathing was probably near 12 percent, but it was not measured.

Prior to testing the water-exposed shearwalls, they were visually inspected. It was found that the OSB surface was much rougher below the waterline than above it as a result of surface strand swelling. Also, surrounding each nail below the waterline, there was a circular “dimple” in the surface of the OSB where the nails had apparently restrained the thickness swelling. The nail heads did not pull through the surface of the OSB. This indicated that the pull-through capacity of the OSB exceeded the forces attendant to thickness swelling. It also showed that the forces related to thickness swelling did not exceed the withdrawal capacity of the nails. On the average, the dimples were 0.8 mm deep relative to the surface of the panel. A result of the residual thickness swelling was that the sheathing below the water line fit tighter to the wood studs after being submerged for 168 hr and re-dried than did the sheathing above the water line or on the control walls.

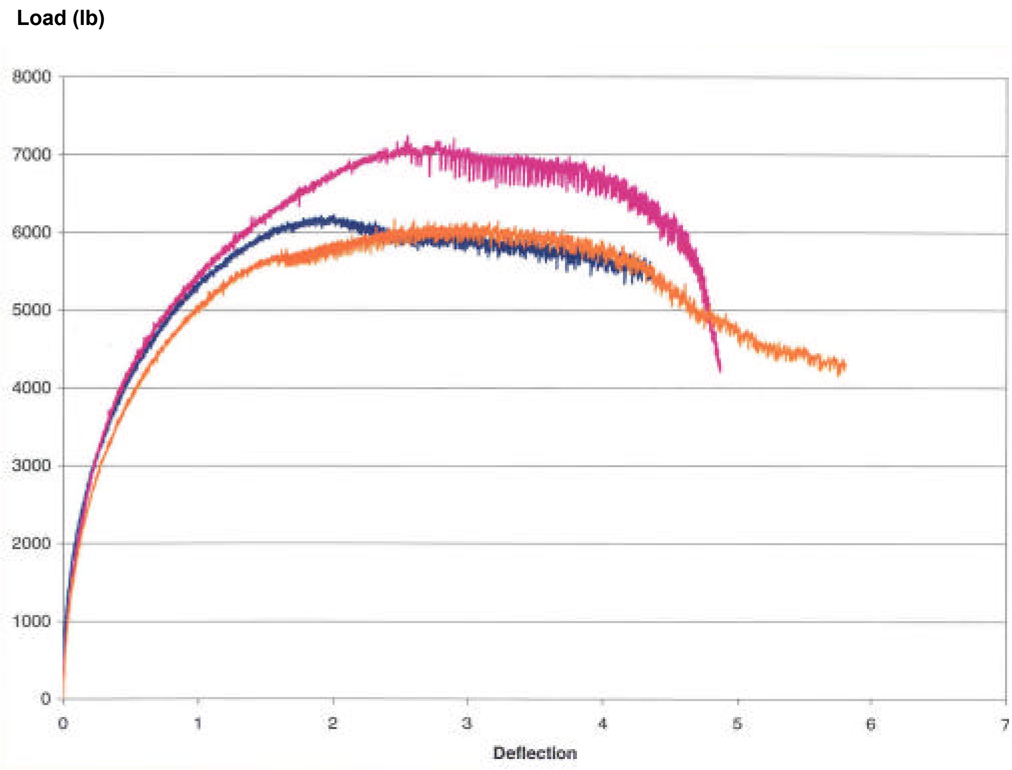


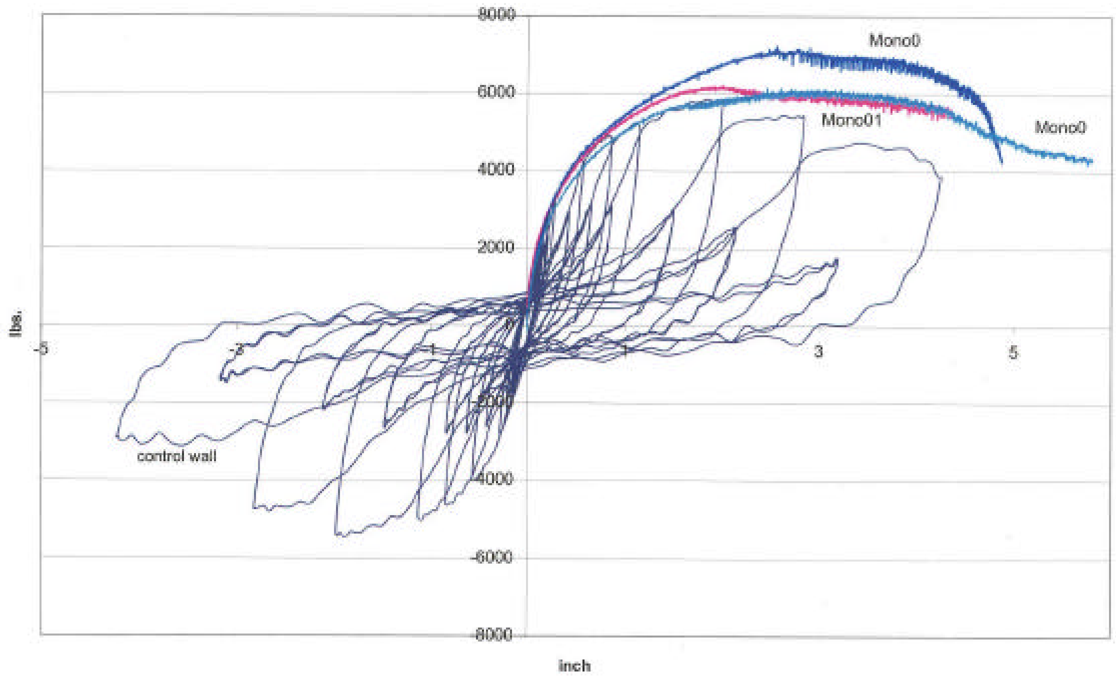

**Figure 4. Displacement-load results for the monotonic tests.**

The results of the quasi-static testing of the control and water-exposed walls are summarized in Table 2. A typical hysteresis diagram for control and water-exposed shearwalls is given in Fig 5. The data indicated that the mean capacity of the shearwalls was not reduced by the water exposure. In fact, the backbone energy and total energy absorption were not reduced by the water exposure. However, the stiffness of the shearwalls was reduced by 27 percent.

**Table 2. Summary of the quasi-static tests of control and water-exposed shearwalls, n=3, where the parenthetical values are standard deviation.**

| Property | Condition | |
|---|---|---|
| | control | water-exposed |
| $F_{max}$ (kN) | 26.8 (3.2) | 30.6 (1.7) |
| $F_{min}$ (kN) | -25.0 (2.1) | -28.3 (1.0) |
| Stiffness (N/mm) | 954 (111) | 696 (57) |
| Backbone energy (kN*mm) | 2405 (335) | 2653 (124) |
| Total energy absorbed (kN*mm) | 11393 (1971) | 13525 (779) |

a)




b)

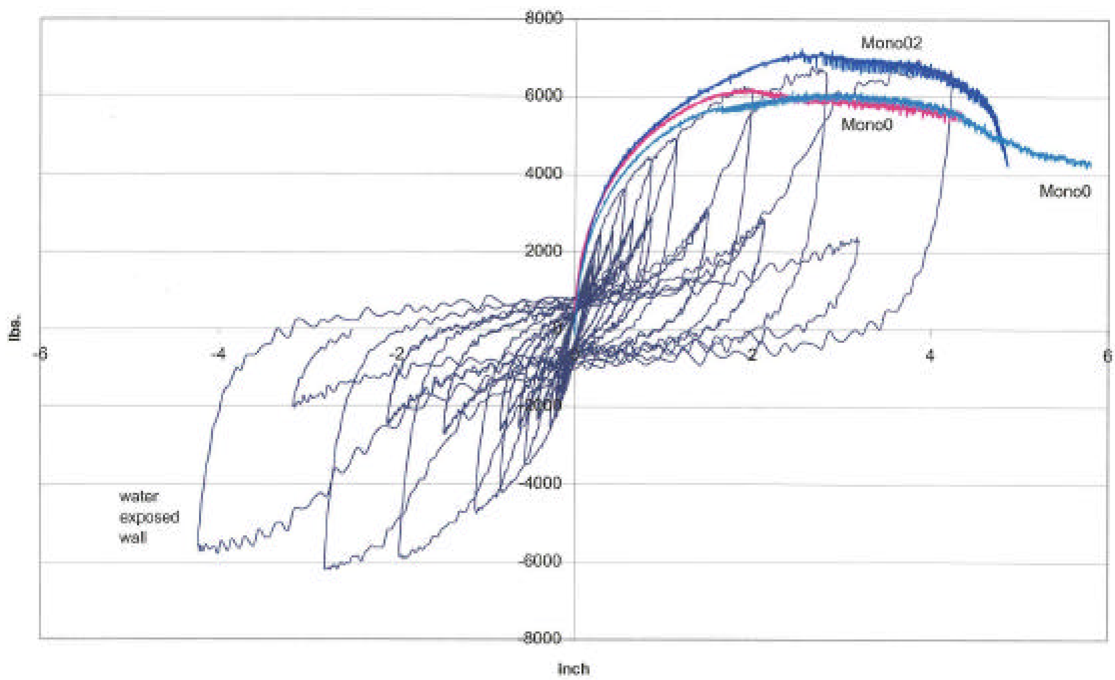



**Figure 5. Typical quasi-static displacement-load response of shearwalls; (a) control, (b) water-exposed.**

The walls failed in much the same manner whether they were the monotonically tested shearwalls or the control or water-exposed shearwalls tested by quasi-static protocol. In general, the wall performance was governed by nail bending and withdrawal (from side grain) capacity. Although, in the water–exposed shearwalls, some of the nails below the waterline were partially pulled through the OSB. This demonstrates that there was a deterioration of the OSB below the waterline. Freitag (2001) conducted a study at Oregon State University with the same OSB and wood to evaluate the effects of wetting and assembly procedures. She reported an increase in the strength of joints when the wood had been soaked. Hence, it is thought that unrecovered swelling may have contributed to nail withdrawal capacity.

The design capacity of the shearwall was based on the design guide (APA 1998), where the design capacity was reduced for the use of 8d box instead of 8d common nails. The diameter ratio for 8d box to 8d common nails was 0.86. Thus, when the wall was new, the design capacity for normal duration was 11.6 kN. The actual capacity of control shearwalls was 29.1 kN. The ratio of design to actual capacity was 2.5.

Given the embedment strength of the OSB as measured, the yield mode for the sheathing nails was expected to be a Mode IIIs as described the NDS (1997). Observation indicated that the nails in the test walls yielded as described by Mode IIIs. As the embedment strength as reduced by 40 percent after water soaking, the yield mode was expected to remain a Mode IIIs, and observation indicated that this was the case. The 40 percent embedment-strength loss translated to an expected reduction in wall capacity of 18 percent based on the yield mode equations, which do not include interlay friction. Because some of the nails in the quasi-static tests of water-exposed walls showed evidence of pull-through, it appears that the shift from a Mode IIIs failure to a pull-through yield mode would occur before any other yield mode can become critical.

A method to calculate the design stiffness of shearwalls is not available because shearwall design is always governed by capacity. In this test, the stiffness of the shearwall was reduced by 27 percent by the water soaking. It is valuable to note that the nail embedment stiffness of the OSB was reduced by 37 percent as a result of the water soaking. Thus, it seems likely that the loss in wall stiffness can be linked to the loss of nail embedment stiffness.

The results of the test program might be different with other sheathing materials or nails; wall performance will depend on the thickness swelling characteristics of the panel and the change in properties with water exposure. For example if the sheathing panel did not swell, then the loss of embedment strength might translate to a real reduction in wall strength.

The 1-m depth of submersion was an arbitrary choice of water depth. Wall height submersion was expected to have only a minor effect on the results of wall submersion tests because the nails at the top and bottom of the shearwall are the most highly stressed. Hence, the test walls could have been submerged only enough to cover the nail connections at the bottom edge of the wall and still produce the same effect as a 2-m submersion. Since most floods involve less than 2 m of water in the structure (Hausmann & Perils 1998), it was felt that a 1-m depth was sufficient to identify the effects.

The walls and sheathing materials tested in this study were air dry. The capacity of the system when wet (saturated) might be much lower than observed in this moisture condition. However, the probability of concommitent major lateral force generating event with the water exposure (other than the force of water flow around or through the structure) is considered to be small.

## 4 CONCLUSIONS

One approach to rational post-flood analysis of building capacity would be to discount the allowable properties of the OSB embedment strength by 40 percent and then discount the wall capacity by a similar percentage. However, the reduction in embedment capacity results in a calculated reduction of only 18 percent. In actuality, the wall capacity and energy properties were not reduced as a result of water exposure. Strength and energy capacity were affected by the connection geometry as well as the interlayer friction, which in this case was increased by the thickness swelling of the OSB. The loss of stiffness was most likely a result of softening in the OSB sheathing, hence the reduction of embedment strength accompanied by a loss of stiffness in the nail bearing resulted in the loss of wall stiffness. The monotonic tests and quasi-static tests failed by nail bending and withdrawal, which demonstrated that the wall capacity was limited by nail bending and withdrawal and not embedment or pull-through characteristics of the sheathing. It appears that structures with OSB sheathing that are temporarily inundated with fresh water and dried expeditiously will have a loss of stiffness but should not experience a reduction in lateral resistance capacity or energy absorption capacity.

## 5 ACKNOWLEDGEMENTS

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# Effect of Mineralogy of Calcium Sources on Geopolymerisation

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**Summary: Recent research has found that geopolymers offer an alternative to ordinary Portland cement. Numerous studies have also shown that a detailed understanding of the mechanisms that govern the geopolymerisation process is essential to assist in developing a new cement-like product which possesses long durability.**

**This paper will focus on one of these mechanisms, which involves the role of calcium in improving cement durability. It is suggested that calcium provides a crucial link between the chemistries of geopolymerisation and the hydration of cement. In essence, this investigation will explore how the different calcium sources affect the process of geopolymerisation. Four different calcium sources were used in this investigation using various sodium hydroxide concentrations to provide the alkaline environment required. They are: granulated blast furnace slag, ordinary Portland cement, wollastonite and calcite. Compressive strength testing and x-ray diffraction were the main techniques used. For the calcium sources investigated, it was found that the difference in CaO content in the source material had little impact on the resultant compressive strength. However, the mineralogy of the calcium sources used and the alkaline conditions present were found to have a significant influence on the resultant product. Using lower hydroxide concentrations, samples prepared with calcium sources which were predominantly amorphous gave a better compressive strength than their crystalline counterparts did. However, when higher hydroxide concentrations were used, samples containing crystalline sources of calcium gave a compressive strength that was at least as good as the amorphous ones.**

**Keywords: Geopolymer; Cement; Durability; Calcium; Mineralogy.**

## 1 INTRODUCTION

Numerous studies have investigated why the ancient concrete is much more durable than their modern counterparts in recent years. It has been observed that calcium silicate hydrate formed as a result of the hydration of modern Portland cement deteriorate while the ancient cement still remain intact under the same external conditions. French scientist, Davidovits (1987) argued that the durability of the ancient concrete was the result of the presence of alkaline aluminosilicates in the structure. Davidovits named this new class of cementitious material as geopolymers.

Geopolymers are a new generation of material with diverse applications in the building industry and waste management. In general, geopolymers are often viewed as three-dimensional amorphous aluminosilicates resulting from the alkali activation of an aluminosilicate source, for example, fly ash. Geopolymeric binders have similar chemical composition as the natural zeolitic materials but without the extensive crystalline zeolitic structure (Davidovits & Davidovics 1988). Davidovits (1991) described the alkali activation process of the aluminosilicate source in terms of a polymerical model. Geopolymers are formed by the polymerisation of individual aluminate and silicate species, which were dissolved from their original sources at high pH in the presence of alkali soluble metals. The products formed are characterized by high mechanical strengths with the following general formula [Eq.(1)].

$$M_n[-(Si\text{-}O_2)_z\text{-}Al\text{-}O]_n.wH_2O \qquad (1)$$

Where M is the alkaline element, - indicates the presence of a bond, z is 1, 2 or 3 and n is degree of polymerization. Theoretically, any alkali and alkali earth cation can be used as the alkaline element (M) in the reaction, however the majority of research has, so far, focused on the effect of sodium (Na+) and potassium (K+) ions (van Jaarsveld & van Deventer 1999). It has yet to be proven whether other alkali and alkali earth cations will participate in the reactions in a similar way. Extensive research has been conducted in the areas of characterisations of geopolymers (Davidovits 1987; Davidovits et al. 1990; Davidovits 1991; Davidovits et al. 1994; van Jaarsveld et al. 1997; van Jaarsveld 2000), improvement of the physical and chemical properties of geopolymers (Phair et al. 2000), immobilisation of toxic materials (van Jaarsveld et al. 1997; van

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Jaarsveld et al. 1998; van Jaarsveld et al. 1999), utilisation of waste materials (van Jaarsveld 2000) and natural aluminosilicate minerals (Xu & van Deventer 2000) in geopolymerisation.

The major difference between geopolymers and Portland cement in terms of their chemical composition is calcium. It is not essential for calcium to be present in any part of a basic geopolymeric structure. However, it is commonly acknowledged that the binding property of Portland cement is due to the formation of colloidal calcium silicate hydrate in a semicrystalline phase (Glukhovsky 1994).

Based on Davidovits' and Glukhovsky's findings, it can be further hypothesised that the formation of geopolymer within the structure is responsible for the durability of ancient concrete. Considering the fact that there are at least 40% by mass of combined CaO and MgO, and less than 5% of combined $Na_2O$ and $K_2O$ in some of the ancient concretes studied so far (Glukhovsky 1994), it is clear that an in-depth understanding of the role of calcium in ancient concrete is essential in order to identify the secret behind the long durability of ancient concrete. The role of calcium in ancient concrete could possibly include: (1) the formation of the traditional calcium silicate hydrate (C-S-H), (2) participation in the geopolymerisation in forming Ca-geopolymer and (3) bridging the bonding between the calcium silicate hydrate and geopolymers or many others. However, there is a lack of documented research involving the chemical mechanisms occurring in such systems. It is probable that both calcium silicate hydrate and geopolymers could be forming simultaneously. As a result, an investigation into the role of calcium in dictating the chemical mechanism will provide answers to the fundamental question as to whether two separate phases will be formed, or a new material will be produced instead.

It is proposed that the addition of calcium to an initially calcium-free geopolymeric system will most likely produce a number of reaction products. The type and number of these products will be dependent on the experimental conditions and, more importantly, the form of calcium present. It is anticipated that one of the most likely products to be formed is calcium silicate hydrate. Figure 1 shows the likely reaction products formed when calcium is added to a geopolymeric system.

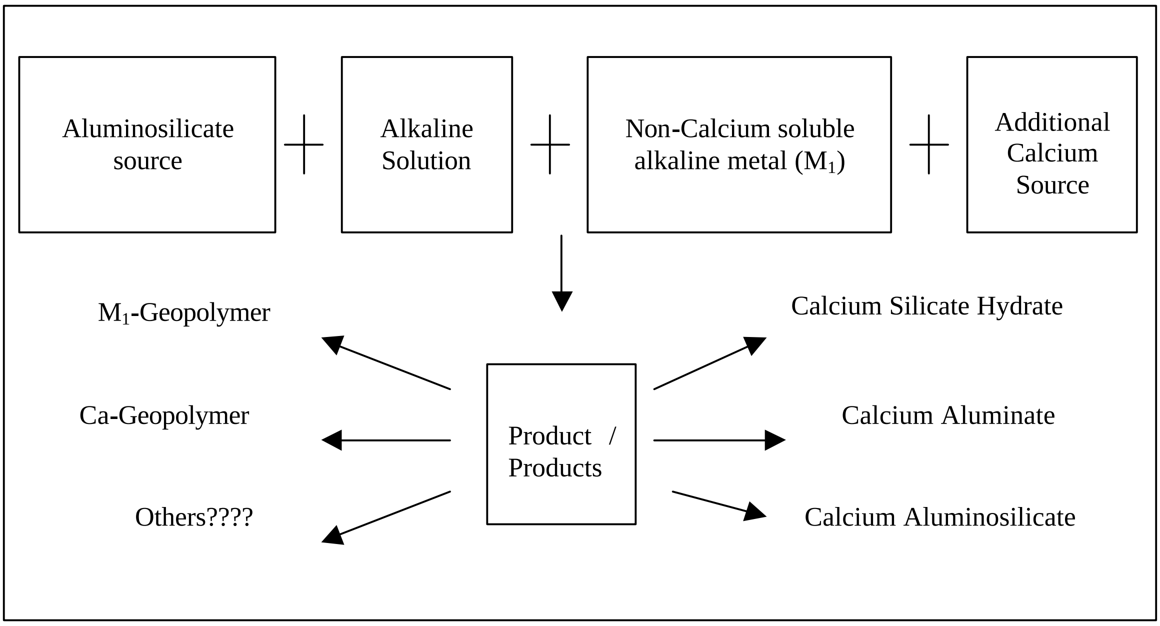


**Figure 1. The concept mapping of the likely products resulting from the alkaline activation of aluminosilicate in the presence of a calcium source.**

The nature of the end product or products formed as the result of the alkaline activation of an aluminosilicate source in the presence of a calcium source is dependent on various factors. These include elemental composition, the mineralogy, physical properties (e.g. surface properties, particle size distribution) of both aluminosilicate and calcium sources, the alkalinity, the nature of the soluble alkaline metal present, as well as the curing conditions and any pre-treatment used. Of greater importance is the fact that it is also influenced by the ratios between each of the raw materials added to the system. It should also be noted that having a high calcium content in the initial aluminosilicate (e.g. flyash) could result in a similar influence on the nature of the final product without any further calcium addition.

Much research has been conducted on the addition of mineral admixture (e.g. metakaolin, silica fume) to a cement paste with the aim of improving the durability and other physical properties of the concrete. However, the majority of such research has focused on the physical effect of the addition of mineral admixture. Additionally, there is a lack of published literature reporting the effect of the addition of mineral admixtures on the chemical mechanisms involved as well as the chemical bonding created or destroyed during the process. Without a detailed understanding of what is actually happening in the process, it will be difficult to proceed further in reinventing a new generation of concrete that possesses the durability observed in ancient concrete.

In this study, a calcium source is added to a calcium-free environment. The aim of this study was to investigate the effect of calcium on the alkaline activation of metakaolin. Four different calcium sources, namely granulated blast furnace slag,

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cement, wollastonite and calcite, were investigated. The mass ratio between metakaolin and different calcium sources, and the $Na_2O/SiO_2$ ratio in alkaline solution were kept constant. The findings from this project are fundamental to any further work involved in the understanding of the durability of ancient concretes.

## 2 EXPERIMENTAL PROCEDURE

### 2.1 Materials

Metakaolin (MK) is a highly reactive metastable clay that is essentially an anhydrous aluminosilicate obtained from calcining kaolin at a temperature range between 650-700°C (Newman 1987). Metakaolin has been used in the past decades as a mineral admixture in cementitious systems due to its capacity for reacting with the calcium hydroxide released during the hydration reaction. In this study, metakaolin was used as the primary aluminosilicate source. This is because Davidovits (1991) suggested that calcination would transform the octahedrally co-ordinated aluminium layers in kaolinite into the more reactive tetrahedral form. This transformation would lead to an improvement in the reactivity, resulting, in turn, in enhanced compressive strength of the resultant binder. Metakaolin used in the synthesis was obtained from ECC International under the brand name of MetaStar 402 with a particle size < 10microns. This commercial metakaolin was purified before calcination, and it contained less than <0.1% CaO as shown in Table 1.

Four different calcium sources (CA) were used in this study, and they are ground granulated blast furnace slag (GGBFS), cement (CEM), wollastonite (WOL) and calcite (CAL). All calcium sources had a particle size distribution of 100% < 50microns. The oxide compositions of the commercial metakaolin and the four calcium sources used are shown in Table 1, which were obtained by X-Ray Fluorescence (XRF) analysis, using a Siemens SRS 3000 instrument.

**Table 1: Chemical Analysis of Metakaolin and Granulated Blast Furnace Slag (mass %)**

| Constituent | Metakaolin | Granulated Blast Furnace Slag | Cement | Wollastonite | Calcite |
|---|---|---|---|---|---|
| $CaO$ | 0.10 | 43.0 | 64.7 | 47.5 | 95.0 |
| $SiO_2$ | 54.78 | 34.4 | 20.45 | 50.5 | 4.0 |
| $Al_2O_3$ | 40.42 | 14.1 | 4.58 | 0.25 | 1.5 |
| $Fe_2O_3$ | 0.76 | 0.11 | 3.72 | 0.2 | |
| $MgO$ | 0.41 | 6.3 | 1.67 | 0.25 | |
| $K_2O$ | 2.72 | 0.33 | 0.39 | 0.04 | |
| $Na_2O$ | 0.07 | 0.30 | 0.67 | 0.01 | |
| $P_2O_5$ | 0.11 | 0.005 | 0.05 | | |
| $TiO_2$ | 0.02 | 0.96 | 0.18 | | |

GGBFS was obtained from Independent Cement, Melbourne, Australia. Cement was obtained from Geelong Cement Limited. Wollastonite (calcium silicate $CaSiO_3$) and calcite (calcium carbonate $CaCO_3$) were purchased from Claywork, Melbourne. These four calcium sources were chosen to cover a range of calcium oxide content in source material ranging from 40% to 95%. These materials also represent different crystallinity and mineralogy, which are readily available in the commercial market.

The sodium silicate solution used in the experiment was supplied by PQ Australia, Sydney under the brand name of Vitrosol N48 (with 28.7% $SiO_2$, 8.9% $Na_2O$ and 62.4% $H_2O$; density 1.37g/mL). Sodium hydroxide pearl (99% purity) was purchased from Orica Australia. Distilled water was used throughout the experiment. Fine washed sand (100% < 2mm) was used as the aggregate in the samples that were subjected to the compressive strength testing. Samples subjected to other analysis were prepared without the addition of the washed sand.

### 2.2 Synthesis

Sodium hydroxide pearl was mixed with the sodium silicate solution to form alkaline solution with two different molar ratios (Ms = $SiO_2/Na_2O$ = 2.3 and 1.5). The hot solution was then cooled overnight to ambient temperature

Samples were synthesised using a MK/(MK+CA) ratio of 0.8. The mass ratio between the dry mix (MK and CA) and the alkaline mix (sodium hydroxide and sodium silicate solution) was 0.69 for Ms = 2.3 and 0.61 for Ms = 1.5. These ratios were selected so as to maintain consistent water content in all synthesised material.

MK and CA were mixed thoroughly until a uniform mix, as far as possible, was produced. The alkaline mix was subsequently stirred with the dry mix to form a paste. The paste was mixed for a further 3 minutes to ensure homogeneity. For samples subjected to the compressive strength test, sand (with mass ratio = sand/(dry mix) = 3) was gradually added to the homogeneous paste until a uniform mixture was formed. The paste was then poured into cylindrical moulds (50mm diameter and 100mm length) and allowed to cure in a laboratory convection oven at 40°C for 24 h before being extracted from the moulds.

The samples which required one-day compressive testing were subjected to the test within 5 hours after extraction from the mould. The remaining samples were left to harden at room temperature (25°C) for a further 6 days (for 7 days test), 27 days (for 28 days test) and 89 days (for 90 days test). All samples were cured at atmospheric pressure.

### 2.3 Analyses

#### 2.3.1 Compressive strength testing

Compressive strength testing was performed as per Australian Standard (AS1012.9-1999) using two 50mm diameter cylinder with a 1:2 diameter to length ratio. All compressive strength measurements were taken as the average of strength results of two samples. All samples were tested after 1, 7, 28 and 90 days. An ELE International Auto Test Compression Machine was used.

The top face of the specimen was cut parallel to the bottom face by using a diamond saw prior to the compressive strength testing. The top surface of the sample was capped with fast setting Boral Dental Plaster to ensure the diameter and length ratio remained at 1:2. The plaster was then left to dry at room temperature and atmospheric pressure for at least 3 hours, prior to testing for compressive strength.

#### 2.3.2 X-ray diffraction (XRD)

X-ray diffractograms were recorded on a PHILIPS PW1800 machine using a Cu $K^{a}$ anticathode and a scanning rate of 2°/min from 5 to 70° 2-theta to give the structural information on each sample. Identification of various crystalline phases was done by comparing the diffraction patterns to JCPDS (Joint Committee on Powder Diffraction Standards) data. The specimens were prepared by mechanical grinding using a ring mill. It should be noted that the washed sand was not added to samples subjected to the XRD analysis.

## 3 RESULTS AND DISCUSSION

### 3.1 Compressive strength testing

The compressive strength of the binders containing different sources of calcium studied at 1, 7, 28 and 90 days is presented in Table 2 for Ms = 2.3 and in Table 3 for Ms = 1.5.

**Table 2: Compressive Strengths of the Binders after 1, 7, 28 and 90 days (MPa) for Ms = 2.3.**

| | Granulated Blast Furnace Slag | Cement | Wollastonite | Calcite |
|---|---|---|---|---|
| 1 | 39.3 | 35.1 | 0 | <5.0 |
| 7 | 47.1 | 47.5 | <5.0 | 7.5 |
| 28 | 54.2 | 53.5 | 18.8 | 25.5 |
| 90 | 58.2 | 55.1 | 24.6 | 30.1 |

**Table 3: Compressive Strengths of the Binders after 1, 7, 28 and 90 days (MPa) for Ms = 1.5.**

| | Granulated Blast Furnace Slag | Cement | Wollastonite | Calcite |
|---|---|---|---|---|
| 1 | 32.8 | 32.1 | 6.1 | 42.1 |
| 7 | 38.6 | 46.3 | 22.7 | 41.9 |
| 28 | 42.1 | 51.4 | 25.3 | 43.4 |
| 90 | 44.5 | 52.8 | 29.3 | 47.6 |

From Tables 2 and 3, it can be observed that the addition of calcium to a geopolymeric binder would improve the compressive strength of the resultant binder with time. It is also clear that the type of calcium source added and the alkaline conditions used have a significant impact on the resultant binder.

By comparing the results from the compressive strength tests on samples prepared under different alkaline conditions (Tables 2 and 3), it was found that alkalinity affects each type of calcium source in a different manner. From Table 2, it is evident that samples prepared with granulated blast furnace slag and cement were double in strength compared to those prepared with wollastonite and calcite when less sodium hydroxide (Ms=2.3) was used after 90 days of setting and hardening. On the other hand, when more sodium hydroxide (Ms=1.5) was added, it is obvious that the samples prepared from the addition of calcite gave the highest earliest strength (at least 10MPa more than the one with GGBFS). Comparing the strength results at 90 days, the strength of the one with calcite was about 5MPa less than the one with cement. It is also interesting to note is that the strengths of binders containing calcite and wollastonite were higher when they were prepared using higher concentrations of sodium hydroxide, while the reverse trend was observed when GGBFS and cement were used.

According to the XRF analysis shown in Table 1, the calcium content of the four calcium sources used in this investigation ranged from 40 to 95% by mass, in which the CaO content in the calcium sources decreased in the following order: calcite>cement>wollastonite>GGBFS. Furthermore, it was not possible to draw a direct link between the calcium content in the raw materials and the resultant compressive strength in the four materials studied. Referring to Table 2 again, samples containing GGBFS had about twice the strength of the sample containing wollastonite at all the times tested. The CaO content of GGBFS and wollastonite were 43% and 47.5% by mass respectively. The 4% difference in CaO content could not have contributed to the 30MPa difference in resultant compressive strength, especially considering cement contains 64.7% and calcite contains 95% of CaO. It is clearly shown that the type of calcium source used would have a significant impact on the resultant product, however it was also demonstrated that the difference in CaO content in the source materials is not the major cause.

The mineralogy of the four calcium sources is vastly different. Slag is amorphous, while cement, wollastonite and calcite all contain different degrees of crystalline phases in their structure. It is anticipated that the mineralogy of the calcium sources would have a significant impact on the resultant product. In order to examine the impact of mineralogy, XRD was used to examine the raw materials prior to alkaline activation and the resultant product after the activation. The results are presented and discussed in the next section.

### 3.2 X-ray diffraction (XRD)

It was found from the compressive strength results that there was a larger difference between the strength of the four samples prepared with a lower sodium hydroxide concentration (Ms=2.3.) As a result, these samples were subjected to further analysis using X-ray diffraction. XRD was used to study the crystallinity and to identify the different phases present in the materials before and after alkaline activation. It should be noted that most of the binders produced in this study could contain a very high percentage of amorphous to semi-crystalline phases. Due to this limitation, it was difficult to correctly identify all the phases present in the resultant structure as specified in JCPDS. Nevertheless, the use of XRD was still useful in providing a general guide by limiting the possible identities of such phase.

The x-ray diffractograms (Fig. 2 to 4) showed the effect of the addition of different calcium sources on the crystallinity of the resultant binders. These figures show the x-ray diffraction spectra of the raw metakaolin and the calcium source used, the dry material mix (80% MK and 20% CA) prior to alkaline activation and the resultant product at 28 days after the activation. The spectrum marked “after activation” refers to the x-ray diffraction spectrum of the resultant binder after the alkaline activation. The spectrum marked “difference” is referring to the difference between the spectra before and after the activation, which was calculated by subtracting “spectra before activation” from “spectra after activation”.

Figure 2 shows the XRD diffractogram of the alkaline activation of metakaolin with the addition of granulated blast furnace slag. The normal scanning range between 5° (2-theta) to 75° (theta) was used in analysing all samples, however Fig. 2 only highlights the section between 20° to 40° because the other regions did not contain any significant information.

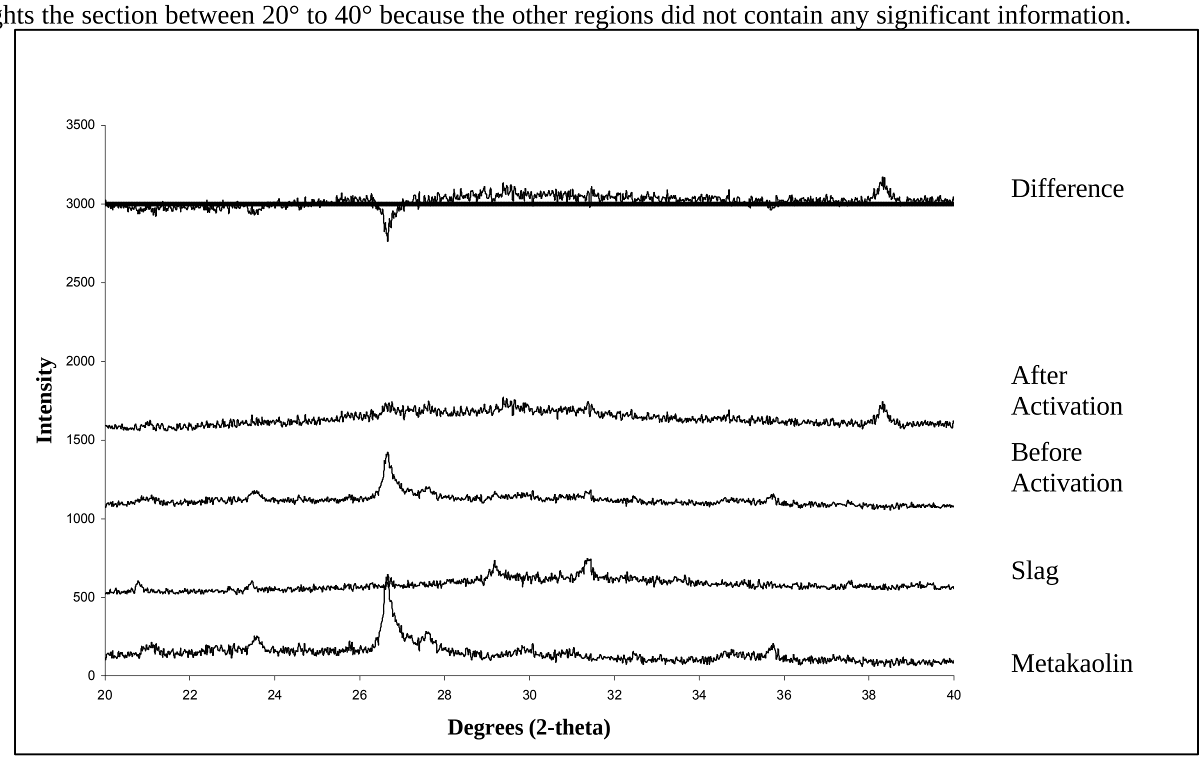

**Figure 2. X-ray diffractogram of the matrix with granulated blast furnace slag.**

XRD analysis of the sample resulting from the alkali activation of metakaolin in the presence of GGBFS reveals the presence of a new but unidentifiable peak (around 38-39 2-theta). The phase associated with this peak was not present in any part of the original spectrum of either slag or metakaolin. Apart from this peak, there was no other new peak found in the spectrum. Figure 3 is the XRD diffractogram of the alkaline activation of metakaolin with cement.

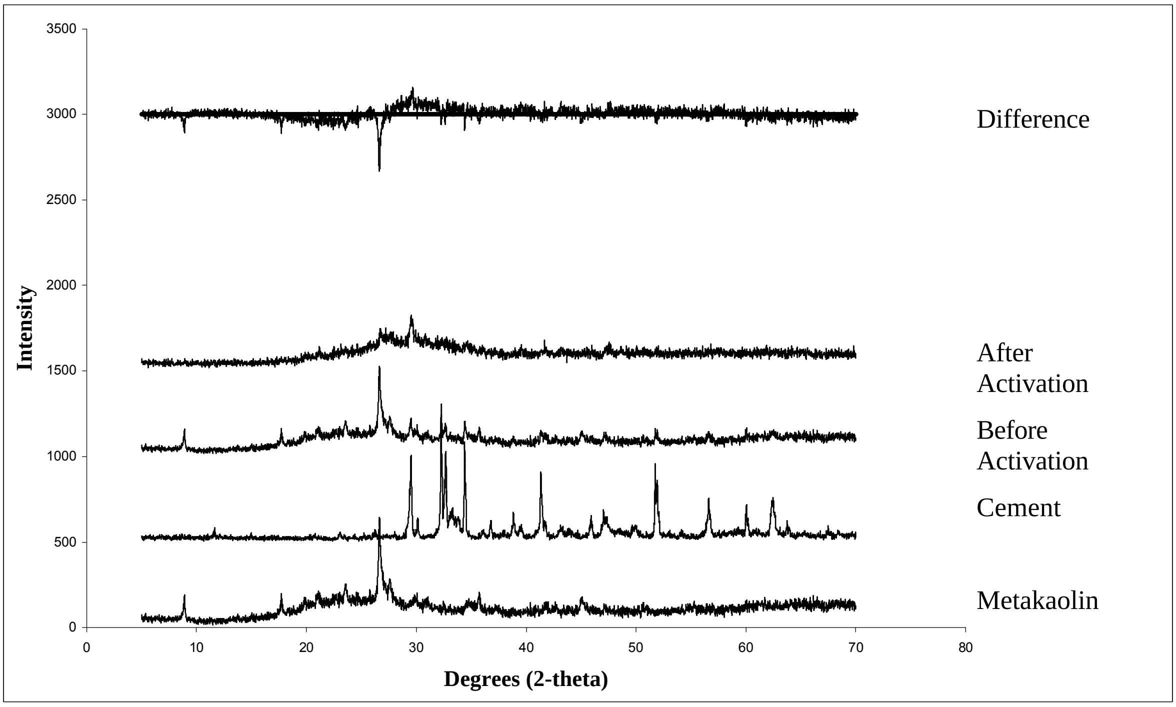


**Figure 3. X-ray diffractogram of the matrix with cement.**

Interestingly, the unidentified peak observed in Fig. 2 does not appear in any of the XRD patterns displayed in Fig. 3 due to alkali activation of metakaolin with cement. This also indicates that there is no crystalline phase forming as a result of the reaction. From the original XRD spectrum of the raw cement (Fig. 3), several distinct peaks indicative of crystalline phases are evident. There was no crystalline phase forming as a result of the activation of metakaolin with cement. In addition, there was also no apparent strong crystalline peak to indicate the presence of significant amounts of undissolved cement particles in the resultant product. Overall, it could be concluded that all of the crystalline phases present in the raw cement had dissolved and reacted in the alkaline solution. This is accompanied by metakaolin becoming part of the amorphous structure of the resultant product. Therefore, it is hypothesised that the calcium present in the cement's crystalline structure had been leached out of the crystalline structure and reacted with the alkali activated metakaolin to form a totally new amorphous calcium-containing material.

Figure 4 displays the XRD diffractogram of the sample obtained from alkaline activation of metakaolin with wollastonite.

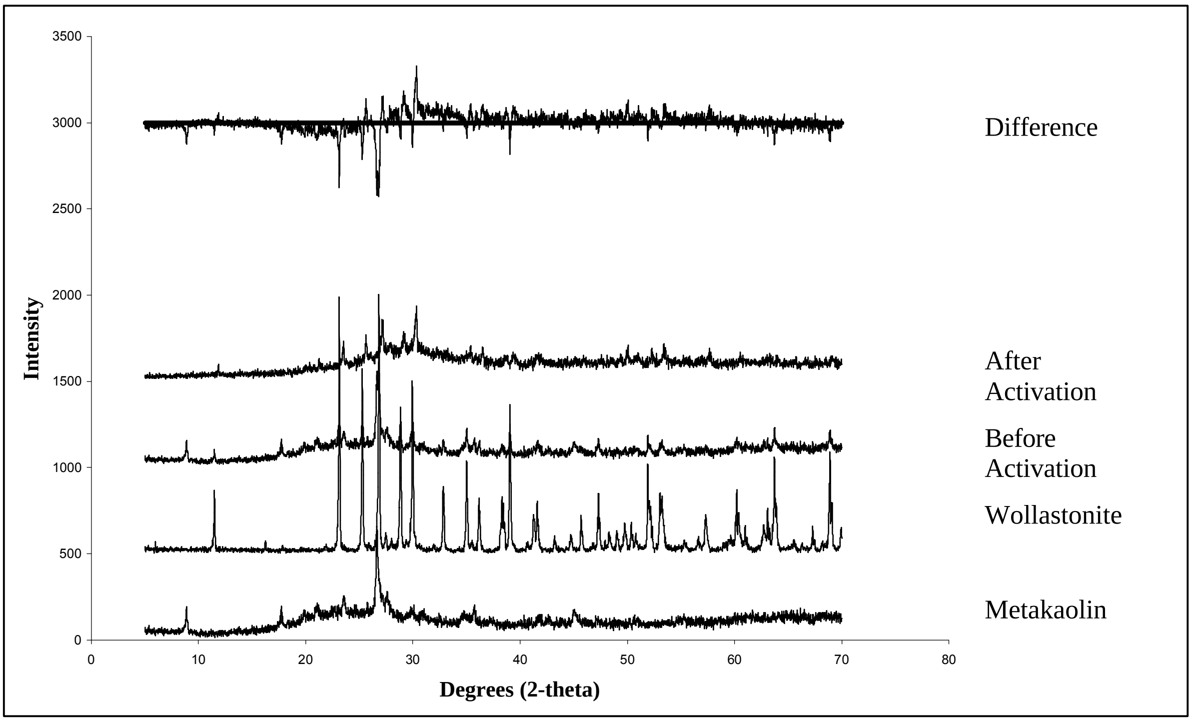


**Figure 4. X-ray diffractogram of the matrices with wollastonite.**

From Fig. 4, it is obvious that, unlike cement or GGBFS, there is still a small amount of undissolved wollastonite particles present after alkaline activation. From the spectrum marked "Difference" on Fig. 4, there exists some undefined new phases (around 20-25 2-theta) due to alkaline activation of the two materials. It is highly likely that these peaks are associated with the undissolved wollastonite particles present. Figure 5 shows the XRD diffractogram of the alkaline activation of metakaolin with calcite.

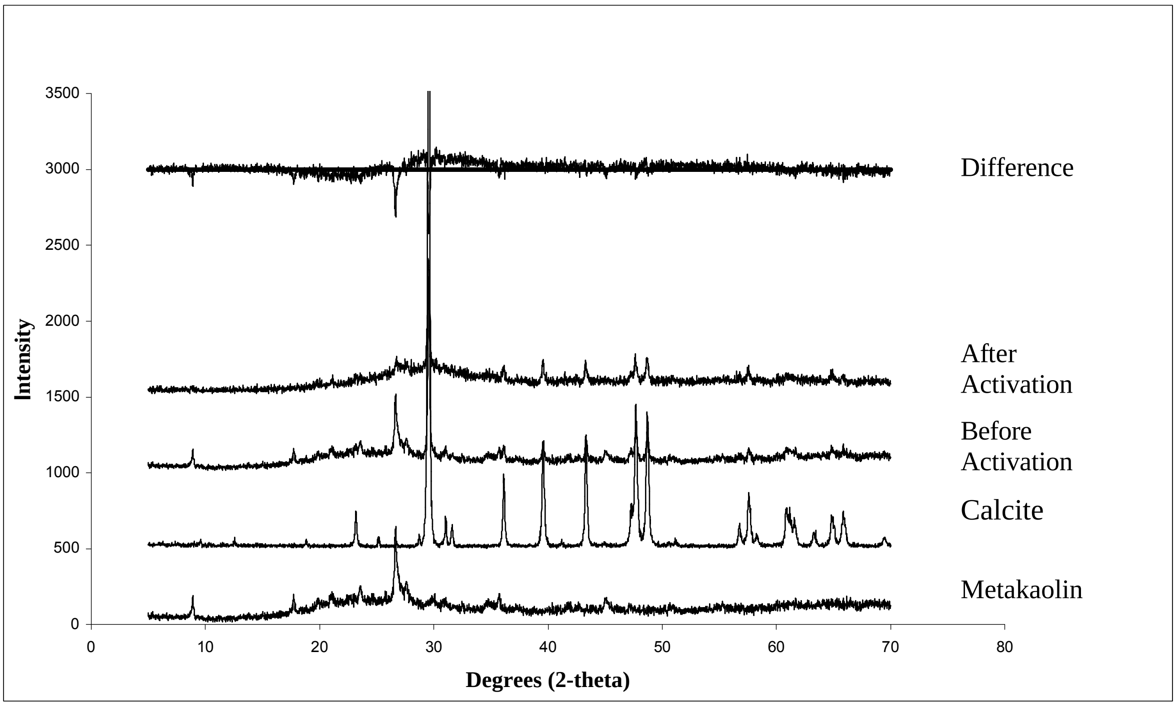


**Figure 5. X-ray diffractogram of the matrices with calcite.**

Like the binders with wollastonite, the XRD spectrum of the binder containing calcite indicates that not all the calcite added to the system was dissolved or reacted during the alkaline activation.

Comparing Figs 2, 3, 4 and 5, the samples with the addition of GGBFS were shown to have formed new crystalline or semicrystalline phase (as indicated by the presence of the new peaks on x-ray diffractogram) after alkaline activation.

However, due to the high amorphous content in the resultant product, it is difficult to identify the new phase formed using XRD alone.

In relating the compressive strength results to the XRD analysis, samples with slag or cement gave the highest strength when a lower concentration of sodium hydroxide (Ms=2.3) was used. The XRD analysis showed that most of the original phases present in GGBFS and cement have been dissolved in these systems, as opposed to what was found in the samples containing wollastonite or calcite. Therefore, it is suggested that high compressive strength could be achieved provided there is a higher degree of dissolution from the calcium sources. The calcium and silicon species present in the original sources would only participate in the reaction if they could be readily dissolved from their original sources. Natural minerals, such as wollastonite and calcite, have limited solubility and reaction in the alkaline solution. This is because the calcium, silicon or even aluminium species were firmly bonded to their original structure. In the case of processed and manufactured materials, such as cement and slag, they can be readily reacted and subsequently participate in the reaction. Therefore, it is proposed that the mineralogy of the calcium sources has a significant impact on the dissolution of calcium, silicon and aluminium from the original source. Following from that, the resultant compressive strength is directly related to the concentration of the dissolved calcium, silicon and aluminium present in the system.

There is another interesting anomaly from the XRD analysis: there was a very weak "hump" appearing at around 28-32 2-theta for all four resultant samples. It is believed that this "hump" is an indication of the presence of an amorphous alkali aluminosilicate in the resultant structure, however it is not possible to establish the identity of the alkaline element without further alternative analysis. Given that there was no new crystalline phase present in the samples with either cement or calcite, it is suggested that the calcium dissolved from the crystalline source material would have reacted with the available silicate or aluminate species to form some amorphous or semi-crystalline materials which could include calcium-based geopolymer, calcium silicate hydrate, calcium aluminate or calcium aluminosilicate. Further work must be conducted in identifying all the amorphous and semi-crystalline phases which could possibly exist in all samples. This information is vital to the overall research into determining the reasons behind the long term durability of ancient concrete.

## 4 CONCLUSION

This work has provided strong evidence implying that high compressive strength could be achieved if there is a high degree of dissolution of calcium, silicon and aluminium from the calcium sources. The calcium and silicon species present in the original sources would only participate in the reaction if they could be readily dissolved from their original sources. Furthermore, it was found in the four calcium sources studied that the CaO content in calcium sources had little impact on the resultant compressive strength. On the contrary, the mineralogy of the calcium sources used and the concentration of sodium hydroxide present were found to have a significant influence on the resultant product. In samples prepared with low sodium hydroxide content, samples with amorphous source of calcium gave a better compressive strength than their crystalline counterparts. Samples prepared using a high concentration of sodium hydroxide and crystalline sources of calcium gave a compressive strength that was at least as good as those registered by the calcium samples sourced from predominantly amorphous minerals.

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# Effects Of Dolomite Aggregate On The Deterioration Of Concrete Floor

**JC Rocha CA da Luz & M Cheriaf**

**Summary**: **There are some aggregates that, when in contact to alkali cations in cements, are highly reactive, such as the dolomite aggregate. This way, even though it presents mechanical strength and low porosity, a material can show deterioration which can cause serious damages to the product over time. This paper presents one case study accomplished in one concrete floor with screed mortar (high strength) in an industry located in state of Santa Catarina/Brazil. This case study shows the formation of gel "unlimited swelling" around aggregate particles caused by effects of alkali aggregate reaction, showing up two years after having been cast.**

**For the case analysis, eight cylindrical samples were extracted presenting the following dimensions: 50mm in diameter and a medium height of 120 mm where 105 mm represented the substratum and 15mm the cement mortar). Besides, scratching and superficial extractions of the cement mortar samples were also accomplished in those visibly deteriorated areas of the floor. The investigation of the case was accomplished using thermal differential analysis techniques (DTA), Scanning Electron Microscopy (SEM) and X-ray diffraction analyses (XRD), where in all of the samples, preparation and operation of the technique were made with extreme severity.**

**Based on the presented results using the SEM determination, it can be concluded that the micrograph of mortar of the analyzed samples of the floor, as well as the samples of the scratching and extraction, presented chemical characteristics morphologies and compositions of the alkali-aggregate reaction. The results of the DTA of the XRD, accomplished in the aggregates used in the composition of the cement mortar of the floor indicated the presence of quartz grains, in the gravel (cheap aggregate) and the white aggregates consisted of calcium dolomite carbonate. Based on the manifestations and the results, the reactive characteristic of the white aggregate and the alkali effects was very noticeable. In the "unlimited swelling", the appearance of the whitish structure is due to the white aggregate disintegration, with strong indication of a reaction belonging to the alkali-carbonate group. The results of analyses showed gel and of the crystallized products, around the white aggregate, presented calcium-silica-alkalis reactions, where the detected alkalis were potassium and sodium. Besides, it was possible to identify, ettringite, portlandite, $Mg(OH)_2$ and carbonate of calcium, which in relation to the gel, the reactive characteristic of the white aggregate was also confirmed.**

**Keywords. dolomite aggregate, concrete floor, deterioration,** ***alkali-aggregate reaction***

## 1 INTRODUCTION

The alkalis usually end up being included to the material (concrete, mortar...) through the cement, but they can also be due to other kinds of sources such as the mixing water where the product of the alkali-aggregate reaction can be manifested even by expansions or pop-outs. So, it is possible for a material, even if presenting mechanical resistance and low porosity, to manifest deterioration which can cause serious damages to the product over time.

According to VEIGA et al. (1999), the reaction alkali-aggregate is done between some mineralogical components and the alkaline hydroxides dissolved in the pores of the product. Based on his studies, the reaction alkali-aggregate that occurs among certain calcareous and the alkaline solutions contained in the pores results in an expansive hygroscopic gel.

In relation to the aggregate types involved in such expansive phenomenon, LEA et al. (1970) say that they are restricted to certain fine-grained soils, clay and dolomite aggregates.

This paper is about a study regarding the reaction among dolomite aggregates and alkalis in a floor of high resistance where the consequences of this resulted in expansions and formations of vesicles with a whitish pigmentation. It refers to a case study in an industry located in Santa Catarina/Brasil, two years after having been cast.

The study was totally based on in situ observations and using microstructural techniques of investigation. The differential thermal analysis techniques (DTA), Scanning Electron Microscopy (SEM) and X-ray diffraction analysis (XRD) were studied. The samples used were cylindrical cores, scratching fragments and superficial extractions of the mortar in visible places of the deteriorated floor, where the sample collection, preparation and operation of the technique were made with extreme severity in all of them.

## 2 MATERIALS AND METHODS

For the investigation of the problems presented by the industrial floor, superficial fragments of mortar (Fig. 1) and 08 test cores (Fig.2) with the use of diamond drill were extracted with the following dimensions:

Diameter = 50 mm;

Medium height = 120 mm, representing the coating mortar (of approximately 15 mm) and substratum of 105 mm.

**Figure 1: Superficial extractions of the mortar**

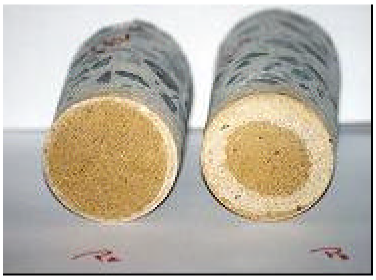

**Figure 2: Cylindrical Test Cores**

It was necessary to visit the place where the manifestations occurred in the floor in order to verify the incidences and any fortuitous conditions of moisture, collecting material for complementation of the analysis.

Having test cores and samples in hand, they were tested using the differential thermal analysis techniques (DTA), Scanning Electron Microscopy (SEM) and X-ray diffraction analysis (XRD).

### 2.1 Differential thermal analysis techniques (DTA)

The analysis were made under the following test conditions:

- constant speed of $10^{o}$C/min;
- natural atmosphere environment;
- calcined caulinate as inert reference material;
- alumina sample carrier;
- Testing mass of 600mg (particles <150µm),
- K type thermocouple and automatic registration of the data during the test in HP’s dataloger.

The test was accomplished in the Núcleo de Pesquisa em Construção of the Civil Engineering Course - UFSC, using the equipment of differencial thermal analysis, endowed with Carbolite oven and manufactured by INSA_Lyon.

Initially, the samples submitted to this test were obtained by the test cores P1 and P8. During the preparation it was noticed that some of the whitish particles in the analyzed material were easily taken to pieces. Because of this, they were divided into smaller samples so that they could be analyzed separately constituting a representative sampling of the manifestation the whitish grains with little mortar adhered to its surface. For each sample two analysis were made.

In Figures 3 to 5, below, the results of the thermal analysis accomplished in the samples are represented.

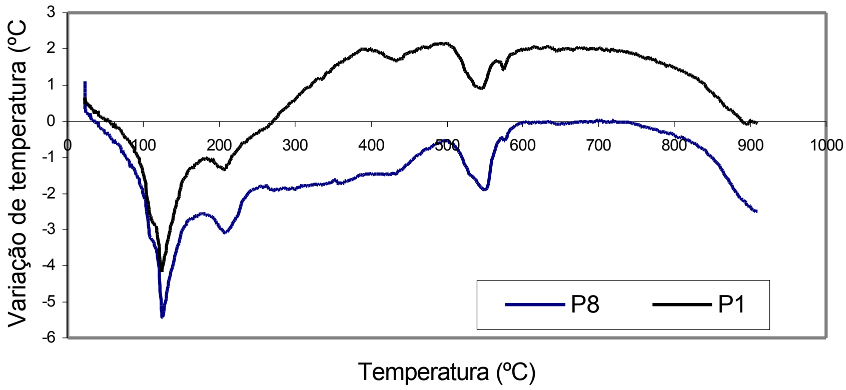


**Figure 3: Thermogram of the mortars of the Test cores P1 and P8.**

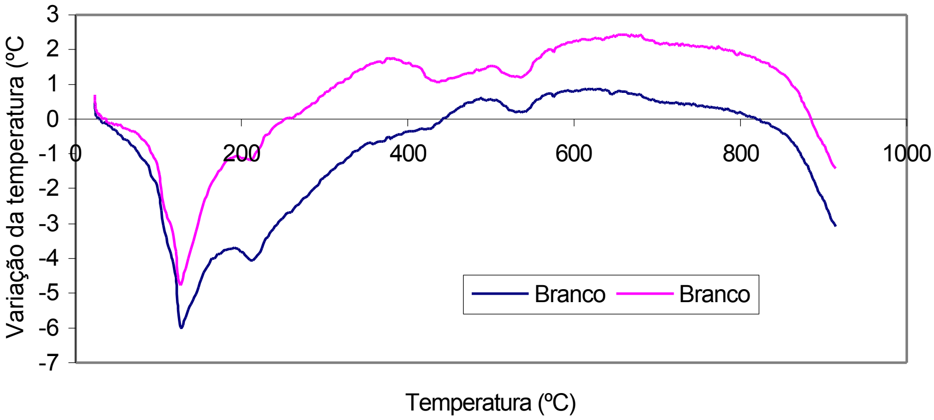


**Figure 4: Thermogram of the white grains with mortar adhered to the surface.**

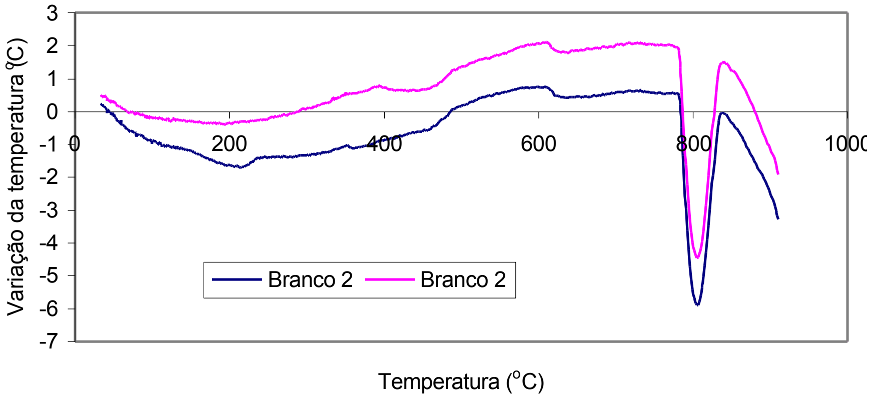


**Figure 5: Thermogram of the white grains contained in the aggregates- without mortar in the surface**

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Of the thermal analyses, based on the peaks, the following components were identified:

- temperature 105-110 °C: loss of water;
- temperature 210 °C: calcium monosulfate ($CaSO_4$);
- temperature 400 °C: magnesium hydroxide $Mg(OH)_2$;
- temperature 570 °C: quartz (Q);
- temperature 800 - 920°C: calcium carbonate CaCO3;

It is important to highlight that the white aggregates showed the predominant composition of calcium carbonate.

## 2.2 X-Ray Diffraction Analysis (XRD)

The preparation of the samples followed the same procedure used in the differential thermal analysis - DTA, that is, the samples were taken to pieces until the desired grain size was obtained. (passing through a 150mm sieve). The analysis were accomplished in the laboratory de Difratometria of the Physics Department of UFSC, using the X-ray diffraction equipment of the brand Rigaku, models Mini Flex, with grafite monocromatizer and radiation Cu Ka.

The obtained results of the analyzed samples are presented in diffractograms shown in Figures 6 to 8. TheTable 1 assists the reader in interpreting the diffractograms.

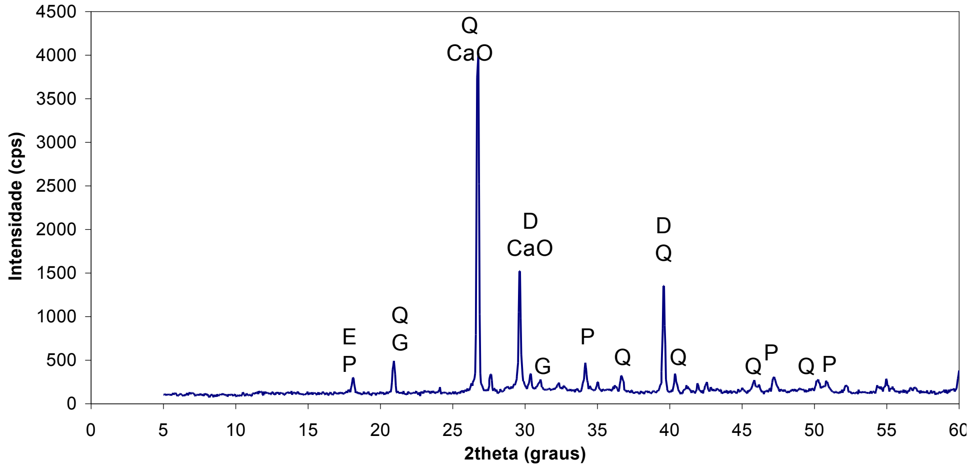


**Figure 6: XRD pattern of the mortar sample of the Test Core P1.**

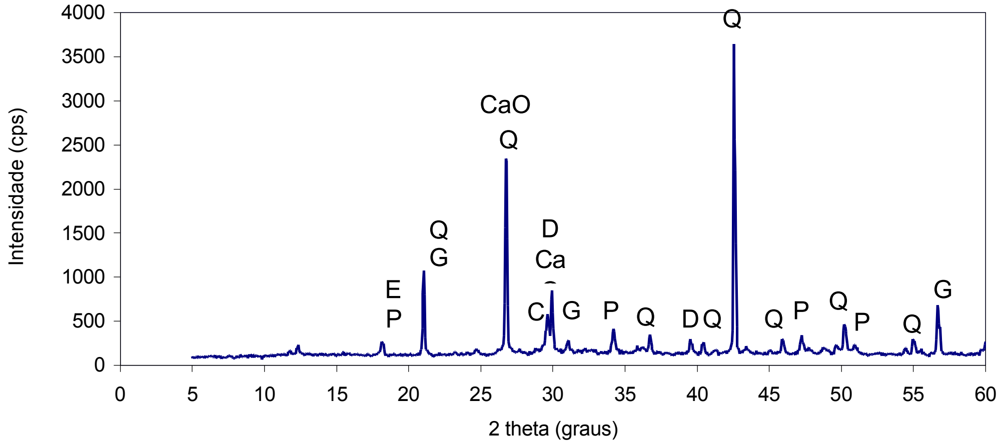


**Figure 7: XRD pattern of the mortar sample of the Test Core P8.**

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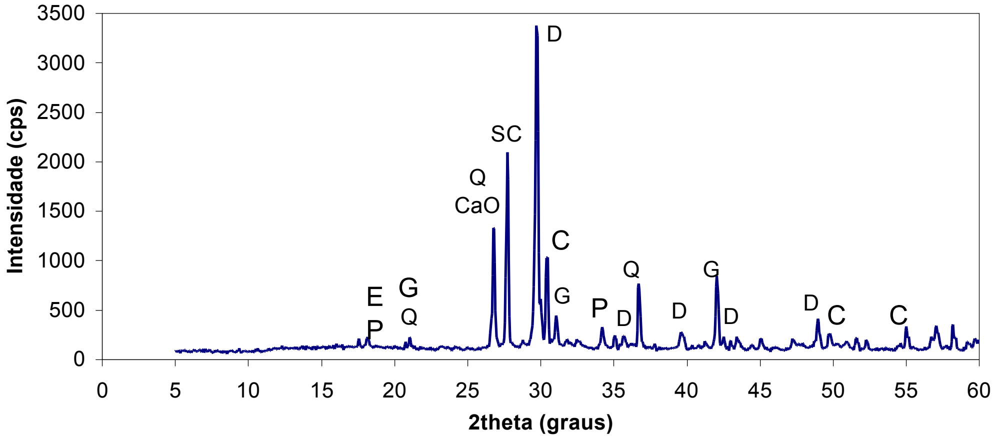


**Figure 8: XRD pattern of the white grains extracted from the Test cores.**

**Table 1: Notation of the minerals**

| Notation | Mineral | 2θ |
|---|---|---|
| Q | Quartz | 21,050°; 26,750°; 36,750°; 39,600°; 40,400°; 42,600°; 45,950°; 50,250°; 50,950°; 55,000°. |
| E | Ettringite | 19,200°. |
| D | Dolomite | 29,700°; 39,650°; 43,450°; 49,000° e 49,750°. |
| C | Calcite | 30,400°; 51,600°; 55,000° |
| P | Portlandite | 18,150°; 34,150°; 47,300°; 50,250° e 50,900°. |
| G | Gypsum | 21,050°; 31,050°; 36,750°; 42,050°e 56,750°. |
| SC | calcium silicate | 27,700°. |

### 2.3 Scanning electron microscopy (SEM)

The scanning electron microscopy (SEM) was used for observation and microstructural analysis of the solid phases (morphology and crystalline lens of the particles). With the same technique the qualitative chemical analysis was used through the microprobe EDX, with a dispersive energy of x-rays (EDX), allowing a located chemical analysis. For the microscopic analysis an scanning electronic microscope - MEV, models PHILIPS XL30, equipped with a micro probe was used. The microscope belongs to the Laboratory de Materiais of the Mechanical Engineering Department of UFSC.

The analysis were accomplished in fracture surfaces obtained through the fragmentation of the mortar of the samples, in points of interest (white aggregates, mortar, pebble aggregate). The samples were prepared covering the surface with carbon coating, accomplished in the equipment BALTEC - EARLY 30 - Evaporad Carbon. Figures 9 to 24 present the micrographs and the spectra obtained through the analysis on the scanning electronic microscope.

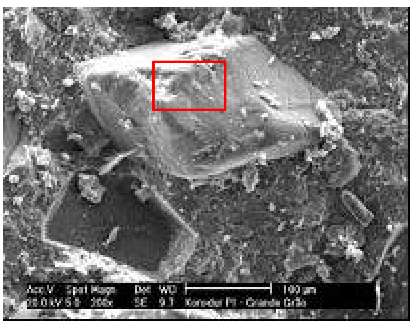

**Figure 9: SEM image of the mortar sample of the Test core P1 (200X)**

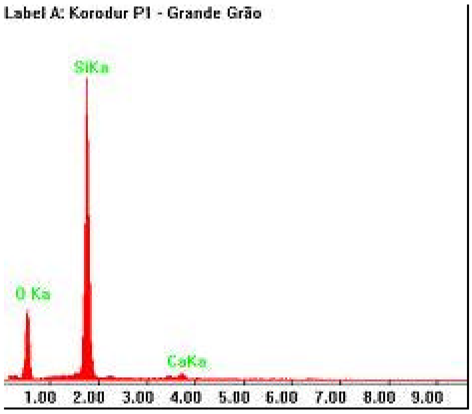


**Figure 10: EDX spectra of the highlighted area in Figure 9.**

In Figure 9 the morphology of the mortar extracted from the test core P1 is observed, with an increase of 200 times, highlighting the pebble aggregates (medium dimension of 300mm), composed essentially of silicon, according to the spectrum on Figure 9. The presence of this component is also identified in the difractograms of the mortars (Fig. 6 and Fig. 7) in a quartz form, as well as in the endothermic transformation to 570° C observed in the thermograms (Fig. 3 and Fig.4).

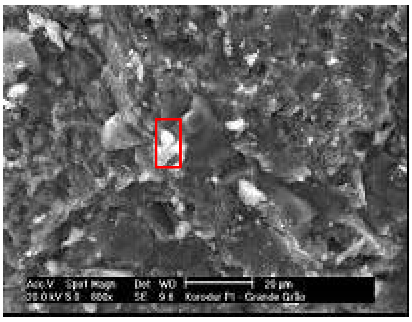

**Figure 11: SEM image of the sample mortar of the test core P1 (800X)**

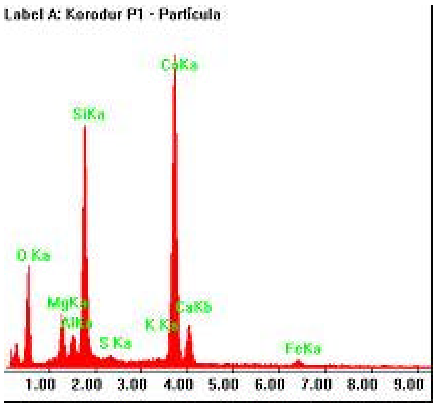


**Figure 12: EDX spectra of the highlighted area of Figure 11**

Figure 11 presents the micrograph of the mortar of the test core P1, with an increase of 800 times, with its referring spectrum in Fig. 12. In the micrograph obtained in the mortar a massive composition it is observed with occurrence of deposits in the surface. The spectrum reveals the composition silicon-calcium-potassium - magnesium, with little occurrence of iron, sulfur and aluminum.

The presence of sulfur can be attributed to the calcium monosulfate (plaster - $CaSO4$), which was evidenced not only in the endothermic reactions at 210°C, shown in the thermograms of Fig 3 and Fig. 4, but also in the difractograms (Fig.6, Fig. 7 and Fig. 8), with a standard peak located in the angles (2θ), presented in Table 1. On the other hand, the presence of magnesium was verified in the endothermic reaction at 400°C (Fig. 3, Fig. 4 and Fig. 5), attributed to the occurrence of dolomite found in the difractograms (Fig. 5, Fig. 6 and Fig.7).

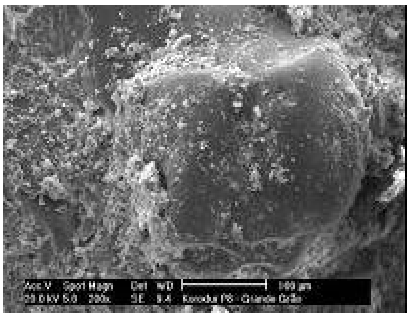

**Figure 13: SEM image of the mortar sample of the test core P8 (200X)**

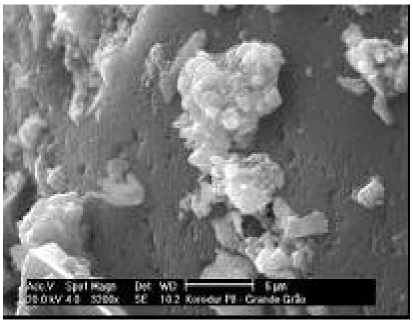

**Figure 14: SEM image of the mortar sample of the test core P8 (3200X)**

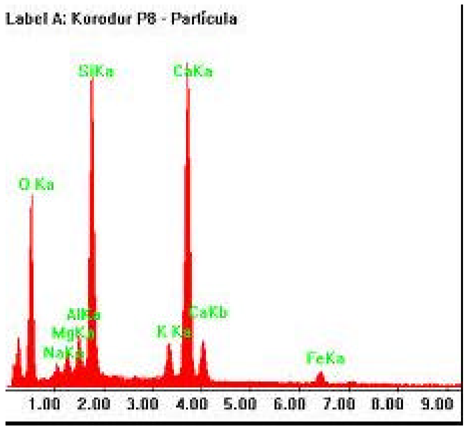


**Figure 15: EDX spectra of the highlighted area of Figure 14.**

Figures 14 and 15 present the micrograph of the mortar of the test core P8, with an increase of 200 to 3200 times, respectively, representing the pebble aggregate inserted in the mortar of the test core. In the micrography of Fig. 14, the aggregate with deposits of crystallized products of the aggregate-alkali reaction in a flocculated structure on the surface is observed, and the spectrum obtained (Fig. 15) reveals silicon, calcium, sodium, potassium, and magnesium components with an occurrence of aluminum and iron.

**Figure 16: SEM image of the white grains with adhered mortar (400X)**

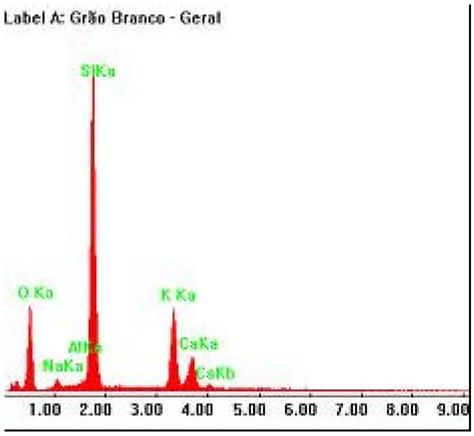


**Figure 17: EDX spectra of the highlighted area of Figure 16**

The micrograph of Fig. 16, with an increase of 400 times, presents the morphology of the white grain with adhered mortar, where the grain and the mortar are dispersed in a gel phase of massive aspect. The spectrum (Fig.17) reveals the silicon, potassium and calcium composition. The a, b and c zones of the analyzed sample, defined in the Fig. 16, had an increase of 1600 times, allowing better identification of the gel formation, of which the micrographies are presented in Figures 18,20 and 21.

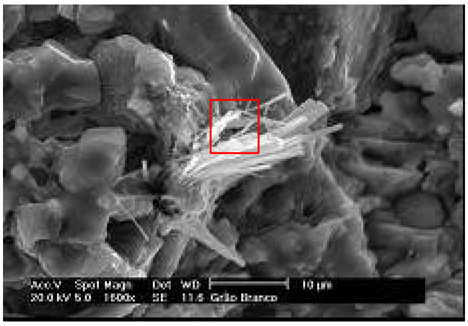

**Figure 18: SEM image of the white grains with adhered mortar (1600X) zone** a **highlighted**

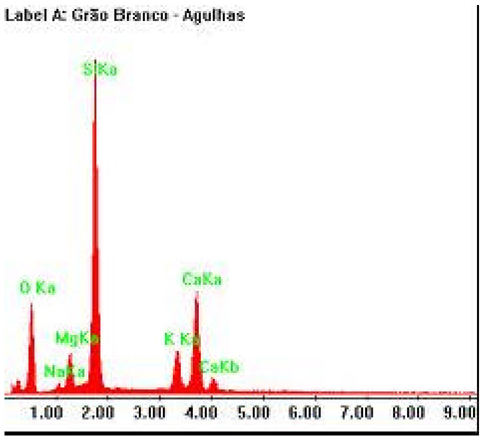


**Figure 19: EDX spectra of the highlighted area of Figure 18.**

In the highlighted zone **a** of the micrograph of Fig.18, the development of the crystalline phase of morphology acicular is observed, composed by silica, calcium, potassium, sodium and magnesium.

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**Figure 20: SEM image of the white grains with adhered mortar (1600X),zone** b **highlighted.**

**Figure 21: SEM image of the white grains with adhered mortar (1600X), zone** c **highlighted.**

The highlighted zone b shows a gel phase containing particles in its surface. In the highlighted zone c (Fig. 21), the internal structure of the gel presents pores. It is still verified the presence of Portlandite, registered in the difractograms of the mortars.

In the samples extracted *in situ*, obtained by the manifestations and observing the mortar adhered to the seal coating, the presence of quartz pebble is verified, indicated by the presence of silicon in the spectrum of Fig. 23. From the analysis of the manifestation morphology, the agglomeration of the product in the surface of the mortar is observed (Fig. 22).

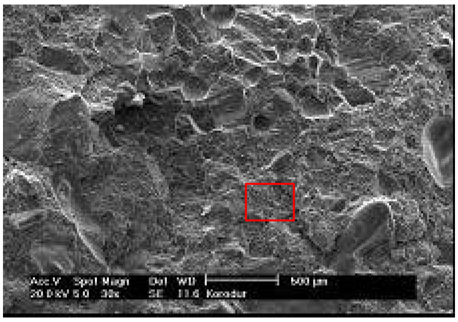

**Figure 22: SEM image in the sample extracted *in situ* (30X)**

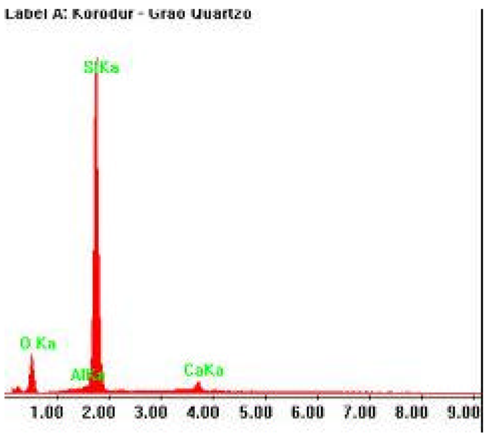


**Figure 23: EDX spectra of the highlighted area of Figure 22 (quartz grain).**

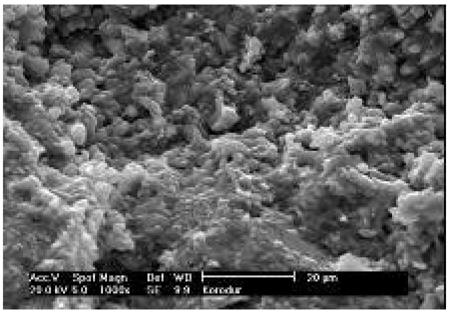

**Figure 24: SEM image of the mortar of the sample extracted *in situ* of Korodur (1000X)**

## 3 ANALYSIS OF THE RESULTS AND CONCLUSIONS

The visit *in situ*, as well as the test cores analyzed showed that the signs of deterioration of the floor were brought by the manifestations observed, marked by the formation of vesicles with a whitish pigmentation and a gel exudation in the surface, indicating alkali-aggregate reaction, which was confirmed by the analysis accomplished in the representative samples.

According to the results presented using the microscope, it is possible to conclude that the analyzed micrographies of the Korodur mortar of test cores of the floor, as well as for the scratching and extraction of the samples *in situ*, presented typical morphologies and chemical compositions of the reaction alkali-aggregate.

The results of the X-Ray difraction thermal analysis, accomplished in the aggregates used in the mortar composition of the floor, indicated the presence of quartz grains in the pebbles and white aggregates constituting dolomite calcium carbonate. According to the manifestations and based on the results, the white aggregates has reagent characteristics to alkalis. In the formation of the vesicles, the appearance of a whitish structure is due to the composition of the white aggregate, with strong indication of a alkali-carbonate reaction.

From a morphological point of view, the gel presented a massive structure in a fine texture with cracking. The crystallized products presented acicular forms, and like the massive gel, in the mortar/aggregate interface. Punctual chemical analysis of the gel and of the crystallized products, done by EDX, showed calcium-silicon-alkaline compositions, where the detected alkalis were potassium and sodium.

Additionally, it was possible to identify, by the scanning electronic microscope (SEM), ettringite in isolated crystals. Well crystallized portlandite was also rarely recognized and identified in the difractograms. In the ones referred to the mortar, the presence of $Mg(OH)_2$ and calcium carbonate was verified, which related to the gel, confirm the white aggregate reativity.

Therefore, in relation to the evidence of the verification of the phenomenon, what should be understood and considered is the alkali-aggregate reaction's risk in the selection of materials for the concrete or any other product that involves dolomite aggregates and alkalis in excessive amounts. This does not mean that the same aggregates cannot be used but that the limitations for use should be evaluated. Besides, it is important to mention that microstructural analysis is always fundamental to determine and to understand the harmful manifestations that usually appear in residential and industrial constructions.

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# Linseed Oil Paint As An Alternative To Wood Preservatives

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**[2]Lading architects & consultants PAR Denmark**

**Summary: For reasons of sustainability great efforts have been made to find environment-friendly alternatives to paints and wood preservatives. This paper presents results from a project where efforts focused on the use of linseed oil paint, which is well known from old buildings.**

**The painting process of using linseed oil paint is quite complicated, and moreover several dogmas attach to the use of linseed oil, whereas scientific evidence is lacking. An investigation was performed in order to see if a correlation could be detected between the composition of the paint, the workmanship and the environmental conditions at the time of application. As it proved difficult to find projects, where sufficient information was available, only 10 projects were surveyed. The result was that no definite answers could be given as to the importance of each individual factor. However, some general conclusions can be given. It appears that the most important factor for a good result was a successful pre-treatment, i.e. the substrate should be dry and in good condition and the linseed oil should be able to saturate the wood completely. Of importance is also the weather at application, that application is performed in thin coats, and that apparently it is necessary to use small amounts of preservatives for the most exposed parts of the window.**

**Keywords: Wood, windows, durability, painting, renovation.**

## 1 INTRODUCTION

Exterior woodwork must be treated with wood preservatives and/or paint to prevent decomposition of the wood due to the action of light and humidity. In Denmark most new windows are vacuum impregnated before they are painted. However, the use of preservatives is not without problems. Much simplified the known products are either not effective enough or they are poisonous or in other ways a strain to the environment. The production of the preservatives may involve extensive use of energy or they might be produced from non-renewable raw materials. Wood preservatives thereby become one of the big environmental problems in building.

Wood preservatives cause problems throughout their life cycle: in the production, in the application, and in the disposal – problems to surroundings as well as work environment.

Till the end of the 1970s wood preservatives and paints in Denmark were almost entirely based on different oils diluted with turpentine or other organic solvents. These products caused problems with the work environment and led to the development of emulsion paints and wood preservatives based on water which was good for the painters but hardly so for the wood. The first generations of these new paints had very poor properties.

These experiences combined with a growing concern about the environment have led to a renaissance of linseed oil paint that has been the traditional product for surface treatment of wood for centuries. Linseed oil has become a popular commodity - the market received it well. However, the products used today are not the same as the "historical products". Among other things the old linseed oil paints contained large quantities of lead, which is a most efficient fungicide.

In reality, very little or no documentation at all exists for surface treatments with modern linseed oil products. In turn many myths have prevailed about linseed oil and its properties. Therefore, the current knowledge of linseed oil paint needed to be summed up and its suitability as wood protection to be assessed.

## 2 THE INVESTIGATION

The main purpose of the investigation was to get an impression of how the products functioned in practice. This was to be done by performing a condition assessment of building parts that had been painted with linseed oil paint and had afterwards been exposed to wind and weather for some years. To be able to draw conclusions from the condition assessments, it was required that:

- The age of the surface treatment should be 5-10 years and preferably no maintenance should have taken place
- Treatment should be linseed oil products only, i.e. no emulsion and alkyd paints etc.
- It should be known what products had been employed
- It should be known how the work had been carried out including the time of the year the treatment was applied.

However, it turned out to be difficult to find suitable objects for examination. This was partly due to the fact that only a limited number of modern linseed oil treatments existed before 1995. Partly because many of the objects are not very well documented or mixed treatments had been used, e.g. alkyd paint used together with linseed oil paint.

Another characteristic is that linseed oil is employed especially in the restoration of historical buildings or by private "do-it-yourself-people". This was reflected in the identified buildings where linseed oil paint had been used namely 6 mansions and listed buildings, 6 cottages at the Open Air Museum plus one single family house and one apartment block. The latter was the only example of an "ordinary building" where painting had been applied in a professional manner.

All the examined products had been applied by brush and all executed on existing woodwork of which some had been mended with new wood.

Some manufacturers have in recent years developed linseed oil paint that can be sprayed on and which is primarily used for factory treatment of new windows. These products have not been on the market long enough to be part of this investigation.

Also, it should be noted that in a lot of cases the products employed no longer exist on the market. Some manufacturers have closed down and others have changed their recipes and ingredients.

**Figure 1. Mending of sashes in old wooden windows in order to preserve as much as possible of the original sashes.**

### 2.1 The surveying procedure

Nine projects were identified which roughly fulfilled the requirements. For each project the following was recorded:

- Identification of the building, its owner and its use.
- The building components treated including wood species and age (original, old, new).
- When and how the surface treatment was applied.
- The product used.
- The orientation of the treated surface and any external conditions of importance.
- Any subsequent maintenance.
- The condition of paint and wood – assessed in the same way as surveyors normally do.
- Possible causes of damage.

The condition assessment is based on a combination of the visual appearance of the surface, e.g. gloss and chalking, measurements of moisture content of the wood (combined with weather conditions, season and orientation) and the physical condition, e.g. whether the wood is soft or hard or whether the paint adheres to the wood or not.

Generally two investigations were performed for each object. First a single person performed a thorough investigation of wood and paint as a basis for assessing the correlation between the present condition and the possible causes for any failures. Afterwards, all projects were further surveyed by two or three persons in order to ensure coherence in the condition assessment.

It should be emphasised that although the employed products were recorded, it was not possible from this project to assess exactly which factors resulted in the longest service life. The reasons were that the number of objects was small, the pre-treatment of the wood is very different, the paints are different and the workmanship is different. It should also be noted that many of the products are no longer on the Danish market either because the factory has closed down or because the recipe has been changed.

## 2.2 Observations Made During The Surveys

The performance of the objects were quite different ranging from good condition after 8+ years exposure to deterioration of paint after about 2 years.

The observations are summarised in the following two lists which are examples of positive observations and problems respectively.

### 2.2.1 Positive observations during the surveys:

- The surface treatment was in good condition and protected the wood well. Probably the reason was that the initial quality of the wood is good and that the workmanship – including the priming – has been performed with care.
- The painting was in excellent condition (after 8+ years) probably due to a combination of high quality of the wood, protection by design, careful workmanship, long drying time between applications.
- Generally the paint was in good condition (after 7 years).
- Woodwork was generally in a good condition even though some flaking of the white paint was found (whereas black and green paint on the same building was still adhering well).
- Woodwork was generally in a good condition even though the lower parts had become soft (start of deterioration). This building is a barn and it is cleaned using a high-pressure hosepipe.
- Woodwork was in excellent condition probably due to a thorough renovation of the windows before painting. Paint adhered well to the surface.
- Generally woodwork was in good condition and paint was adhering well (6 historical houses in the Open Air Museum). Generally the paint and the maintenance appears to be effective. There were no marked differences between older and newer treatments probably because the woodwork was maintained well (every 5 years with a mixture of turpentine and linseed oil).

**Figure 2. Linseed oil painted door with good performance after 8+ years. The wood quality is good and the workmanship of high quality.**

2.2.2 Problems encountered during the surveys:

- The fittings were the most vulnerable parts of the objects as they were not sufficiently protected by the linseed oil products. This is a remark generally valid for most of the surveyed buildings.
- Windows in upper floors showed most chalking probably due to a greater exposure to sun. The paint film was quite unacceptable – appeared like crocodile skin – probably due to evaporation of thinner after the linseed oil had dried.
- The surface was relatively susceptible to dirt.
- The paint film had deteriorated and had not protected the wood sufficiently.
- The paint film was deteriorated and did not provide sufficient protection of the wood. This condition was achieved in a couple of years after painting and is probably due to imperfect priming, wrong succession of the applications of paint (thicker paint applied before thinned paint), too thick layers, insufficient drying between applications.
- The painting was deteriorated and has not provided sufficient protection of the wood (the surface treatment was 11 years old and had not been maintained). The paint was flaking. The mending of the wood had deteriorated somewhat. The reason for the insufficient durability was not clear as the surface treatment and the workmanship was not well described.
- Prevalent chalking of paint due to a combination of too much filler, too thick layers of paint, application in cold weather.
- Not all pigments were stable which was reflected e.g. in the colour of the green paint that varied from almost black to turquoise-green.

**Figure 3. Bad performing linseed oil paint after about two years of exposure. Probably the problems encountered are due to insufficient cleaning of the window before application combined with poor workmanship and unfavourable weather conditions at application. The photo also demonstrates that linseed oil does not provide sufficient protection to sash-angles and other metal parts.**

## 3 GENERAL RESULTS

As mentioned earlier it was not possible from this project to assess exactly which type of paint had the longest service life. However, it was possible to derive some general results from knowledge about the products used, the application conditions, the orientation of the surface treated surfaces and the observations from the surveys. Also general knowledge from earlier projects (Brandt, E. 1999), (Lading, T & Brandt, E. 2000), (Hjort, S. 1995) etc. was taken into account when assessing the results. The results are given below divided into factors considered most important for the durability

### 3.1 Product imperfections

There is a pronounced difference in the quality of linseed oil products, considered to be due to the fact that there is generally too little knowledge of the factors affecting the properties of linseed oil. Also a number of product imperfections suggested that not everybody in the trade had the necessary professional background and expertise:

- Lack of quality control and homogeneity in the production
- products from the same manufacturer, which ought to be alike, may vary in properties and quality from batch to batch
- Too much or wrong filler
- filler was added partly to make the paint fuller (which means that the viscosity is enhanced to make the application easier), partly to make it cheaper. Too much or wrong filler will result in chalking, wrong viscosity etc. There were examples of thinner being needed because too much filler had been used in the first place!
- chalk is often used as a filler, but along with the subversion of the linseed oil it might become visible as chalking; the surface becomes lustreless and gradually gets a white, foggy surface; at the same time it becomes more susceptible to humidity – this was especially distinct upon dark surfaces.
- Lack of production tests
- usually manufacturers of paint and varnish save test samples of the production so later examination is possible in case of problems - some suppliers of linseed oil products were not in line with that.
- Lack of knowledge of basic chemistry
- when marketing a thinner for linseed oil paint that takes longer to evaporate than the linseed oil takes to dry.
- further, there have been examples of products, e.g. containing boron (used as a fungicide), introduced as totally harmless and almost eatable in spite of the fact that boron is suspected of giving embryonic damages, and that it is on the list of substances that should be phased out.
- Contents of unhealthy substances

- especially organic solvents such as turpentine which some products contain to a larger or smaller extent – and which is usually not necessary
- Lack of documentation for products that are sold in connection with linseed oil
- e.g. some manufacturers introduced citrus seed oil as a natural and harmless fungicide with no documentation of the correctness of this and without information of possible unhealthy side-effects.

It should be stressed that also a lot of good products are found on the market. The problem is the difficulties of finding out which products are good and which are not.

**Figure 4. Repair of linseed oil painted window where the paint used for the repair differs significantly from the original due to a larger content of chalk filler.**

### 3.2 Fungicide

The only linseed oil products on the Danish market with fungicide against dry rot are primers, which also might contain other oils and turpentine. It was not possible in all the investigations to find out whether the woodwork, apart from having been treated with linseed oil products, had also been treated with primer oil against dry rot.

Woodwork treated with fungicide against dry rot is probably, other things being equal, in a better position to resist a wrong application than wood with no treatment. Conversely, a correctly performed treatment, where priming with oil has been properly carried out, will probably provide sufficient protection. Humidity will not be at such a high level that mould and dry rot will develop. However, it is important to maintain the treatment. The linseed oil paint (the topcoat) should preferably have fungicide against mould and mildew added in order to prevent growth of fungi.

There is a potential in limiting the use of fungicide to those places only, where it is necessary instead of treating all woodwork (as done with vacuum impregnation). Normally, it is easy to predict where the risk of is, e.g. 95% of the rot attacks in windows are found in the bottom part of sashes and frames.

### 3.3 Application

The application condition and the application process are crucial for achieving a good result.

The protection of the wood is based on the combined action of the primer and the paint (topcoat).

The function of the topcoat is twofold namely to protect against water as well as against degradation from UV-light. However, the most important factor for the protection of the wood is the priming that protects against humidity even when the topcoat is degraded. In order to protect the wood the primer should saturate the upper layers of the pores in the wood completely so that there is no "room" for liquid water. To assure a sufficient saturation it is therefore important that the wood is dry (max. humidity 12-15%) when the primer is applied.

The linseed oil paint should be applied in thin layers and one layer must be thoroughly dry before the next one is applied. If paint is applied in thick layers, it wrinkles and gives the surface the look of crocodile skin. Also the paint is not solid until it has dried up and consequently it is more susceptible to destruction until it has dried completely. In cold weather, i.e. below 5-10° C, it is difficult to apply thin layers and drying becomes slower. The same occurs in damp weather. Therefore, generally skilled users are required to obtain a satisfying result, as it e.g. is a matter of experience to apply a coat of paint in a thin layer and to assess when the previous layer is sufficiently dry to proceed.

**Figure 5. Paint looking like crocodile skin due to application in too thick layers.**

## 3.4 Logistics

Linseed oil treatment does not fit in very well with the logistics at a modern rational building site – especially not if other work is to be carried out at the same time. Long drying hours and sensitivity to dust are examples of the factors that make the use of linseed oil products difficult.

The most durable of the surveyed treatments had been performed most carefully: after priming the windows, 5 coats of linseed oil paint were applied with at one-week intervals, that is, a total duration of more than 6 weeks. Such a long time interval will be hard to fit in with ordinary building procedures, e.g. within urban renewal.

Preferably, linseed oil painting should be performed in a workshop. If the treatment is performed on the spot it should, preferably, not be performed simultaneously with other work. The need for scaffolds in an extended time because of the long drying hours further increases the price of using linseed oil products.

## 3.5 Potential

It may be possible to make use of the good properties of linseed oil and still take into consideration logistics and application criteria if linseed oil is used as a primer and a 'modern' paint (e.g. aqueous alkyd) as topcoat. The paint should preferably be marketed in *systems* to ensure compatibility and product guarantee.

A good linseed oil system should include:

- An all round primer based on heat-treated linseed oil.
- A special primer with fungicide against dry rot for treatment of the most exposed areas.
- A 'modern' topcoat paint that has sufficient adherence to the primer and that dries fast

### 3.5.1 New windows

New factory-made windows treated with linseed oil products are marketed but are more expensive than similar windows with ordinary paint. This may be compensated for by its durability that eventually will result in a better over all economy – but as far as it is known, no actual documentation for improved durability exists.

A new windows treated linseed oil should have:

- An accelerated ageing test to assess the durability of new industrially treated windows.
- Documentation of performance of factory applied paint systems including durability.
- A calculation of the overall economy compared to traditional windows.
- A labelling scheme to make it easier for the builder/user to compare products including their impact on the environment.

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## 4 CONCLUSIONS

With linseed oil products it is possible to obtain a good protection of wood as well as a durable result, but things may also go awfully wrong. Some of the surveyed treatments proved to have excellent durability – others absolutely not. In some cases the wood was in good condition even though the paint film did not look very well. In turn, no examples were seen of wood in a poor condition under an almost intact coating – something that may be seen in connection with e.g. latex paint.

On the whole, too many imperfections were found even though the investigation only included a relatively moderate number of treatments. The imperfections were found in both workmanship and products.

In all surveyed examples (except a barn in Vemmetofte) professional technical consultants had taken part in the construction work – also consultants specialised in restoration. Still, there turned out to be big differences. This was mainly due to the quality of the wood, the products and the application.

There are several reasons for linseed oil treatment not having been used on a large scale outside restoration of historic buildings and private use.

- Good linseed oil products can be found on the market but not an optimal system. Surface coatings are normally found as *systems* which include products for priming, intermediate coating and top-coating. Similar systems are not found for linseed oil products, which make it more difficult to use as compatibility of the different products must be assessed individually. It is obviously difficult for both consumers and professionals to get the right information and to choose the right products. Differences in quality often do not become evident until after several years, and the purchase of linseed oil products is therefore a matter of trust.
- There is a lack of documentation and product development
- In general it requires skilled users to obtain a satisfactory result, as e.g. it is a matter of experience to apply a coat of paint in a thin layer and to assess when the previous layer is sufficiently dry to proceed.
- The current linseed oil treatments do not fit very well with the logistics on a modern building site especially because of the long drying hours.

In conclusion the prerequisites for obtaining a good result are:

- The wood must be sound and dry.
- The weather must be 'good' during application – i.e. low RH and a suitable temperature.
- The products - primer and paint - should be of a good quality. Product development is needed.
- The priming should saturate the wood completely with linseed oil in order to protect the wood against liquid water and thereby fungus and dry rot.
- The application must be correct - several thin layers that are allowed to dry sufficiently in between applications.
- Maintenance should be performed in due time.

## 5 ACKNOWLEDGMENTS

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# Experience With Equipment For Large Scale Accelerated Ageing Tests

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**Summary: Artificial ageing of building materials and components is a very complex matter. Many factors have to be taken into account including a number of ageing factors and the size and geometrical shape of the test specimen. Most equipment for accelerated ageing is designed for small samples which, however, has the disadvantage that it is not possible to test components including joints in full size. In Denmark and Norway large-scale test equipment has been in use for more than 30 years and its use has been described in test methods. One of these has recently been revised and issued as a Nordtest method based on experience from its use in Denmark and Norway. This paper describes the design of the test equipment including the background for selection of type and size of the ageing factors. Further experience from the use of the method is illustrated by examples of test results for different materials and components.**

**Keywords: Accelerated ageing, ageing factors, deterioration, degradation, durability.**

## 1 INTRODUCTION

Great interest focuses on the service life of building materials, components, and assemblies, which has a decisive influence on the annual cost of buildings and engineering works. Moreover service life has great influence on the environmental impact from materials and components. However, estimating service life of building materials, components and assemblies is a very complex matter and over the years several ways of achieving this goal have been proposed. For well-known materials, knowledge about service life can to a great extent be based on experience. For new materials and components, no experience exists and consequently information about service life needs to be achieved in other ways. According to the RILEM recommendation (Masters & Brandt, 1989) this can be done either through natural ageing, e.g. by long-term exposure under in use conditions, or accelerated ageing at exposure to severe climate, e.g. in deserts. Alternatively artificial ageing may be used. When designing an artificial ageing test, many factors have to be taken into account including not only the ageing process, i.e. whether ageing should be achieved using simulated severe climates or by specific enhancement of selected deterioration factors such as UV light, heat or freeze/thaw, but also the size and the shape of the test specimen.

In Denmark and Norway large-scale test equipment has been in use for more than 30 years and the use has been described in test-methods. One of these has recently been revised and issued as a Nordtest method based on the experience from the use up till now.

## 2 DURABILITY

Often the term durability is being interpreted as the same as service life. In this paper durability is used as a term for the time related capability of a material or component to resist deterioration factors that may reduce its service life. In this definition durability is a property of a material or component, but it does not give any meaning without a precise description of the methods used for determining the durability. Methods for the accelerated ageing used and the methods for recording changes, if any, must both be clearly defined. In this respect durability is in principle not different from other properties, e.g. tensile properties, which give different results if measured at different temperatures (climate) and/or at different rate of elongation (test method). The durability may then be defined as the time of exposure to the artificial climate/climate cycle before a certain change in the property recorded is achieved.

*The durability may then be defined as the time of exposure to an artificial climate/climate cycle before a certain change in the property recorded is achieved.*

When the durability is determined for a material or component, it may be used for estimating service life under natural ageing in practical use. The natural ageing is the result of the combined action of all the ageing factors involved. When trying to estimate service life it should be remembered that the climate is not a constant, i.e. the weather not only from one place to another, but also in a specific place varies from one year to the next. Further the exposure of a material or component varies

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with its position in the building (whether protected or not) and with its orientation. Finally the resistance to natural ageing is dependent on the workmanship performed when mounting the material/construction. Service life of building materials and components thus depends on time and place, construction and workmanship and cannot be given with great accuracy.

## 3 ARTIFICIAL WEATHERING

When performing artificial weathering all ageing factors of importance for a specific material or component should ideally be increased by the same factor, i.e. the acceleration should be the same for all crucial degradation factors, and they should be acting together in the same way as during ordinary use. If the factors are acting individually, the result may be quite different than if the factors are acting together. When performing artificial weathering in a laboratory it is next to impossible to achieve such a balance between the ageing factors. Instead one seeks to concentrate the individual climatic factors so that in total they produce a cycle of strains that gives degradation results similar to natural exposure but in a much shorter period of time.

The most important degradation factors are normally considered to be:

- *UV radiation from sunlight* resulting in deterioration of chemical bonds, also in combination with humidity.
- *High temperatures (mainly due to sunlight)* increasing chemical degradation processes.
- *Low temperatures* especially *due to winter conditions and to irradiation to the sky* resulting in vulnerability to mechanical impacts.
- *Temperature changes* result in differential movements between different materials and thereby strain in the materials. *Freezing point transitions in combination with humidity over the critical limit* may break porous materials.

Other impacts may also be of great importance:

- *Precipitation/rain* is contributing to the degradation of materials by supplying water that may be used in symbiotic degradation together with e.g. UV radiation or freezing point transitions. Also *leaching* by water may cause degradation.
- *Effects from wind, particles and chemicals* (pollution) may contribute considerably to degradation.

The importance of the degradation factors depends on the material and/or the component studied e.g. UV radiation may be important for plastics but not for concrete and conversely freezing point transitions may be important for concrete but usually not for plastics.

Accelerated ageing tests in a laboratory always carry the risk that the deterioration of the material may be dominated by other degradation mechanisms than those that will dominate in natural ageing. Since this possibility does exist, the results of the exposure should as far as possible be compared with long-term natural ageing performed with the same materials or similar constructions. However, due to the combination effects the endeavours to accelerate the individual ageing factors to the same extent should not be exaggerated. The important objective is to compose an ageing cycle that gives reasonable correlation with results obtained by natural ageing. In this context reasonable correlation means that for a specific material the ratio between the important acceleration factors is the same.

In the Nordic countries low temperature and freezing point transitions are considered to have an important impact on the service life of many building materials for outdoor use. For some materials the freeze/thaw situation during autumn/winter/spring may actually be the most demolishing situation. An illustrative example is brickwork, which after a short exposure time might start to flake off material if it is not suited for a Nordic climate, i.e. wet and with frost transitions. Therefore all the large-scale equipment for artificial ageing presented in this paper includes exposure to frost during the climate cycle.

### 3.1 The use of results

Results from artificial weathering may be used to determine a reference service life in accordance with the definition of durability given above. A prerequisite for this is that the ratio between the acceleration factors found for comparable building materials are the same independent of their practical use (climate/construction). Such “scientific” comparisons have been done also with large-scale equipment as described in this paper. The results show accelerating factors of from 12 – 20. This gives the possibility of a rough assessment of the service life of building materials.

However, when performing artificial ageing, the results are, for several reasons, rarely given as some sort of service life. One reason is that more often the possibility of comparing the results with in situ exposure does not exist. Another reason is that so many factors other than the durability, as defined above, are important for how long time a material may function according to its intended use. The “scientific” comparison of artificial weathering and natural weathering therefore has limited value as a general tool for estimation of service life.

In some cases, e.g. in connection with technical approvals, the results are used to assess the service life of a specific product, whereas in connection with development of products, comparison or ranking of samples of varying compositions may be the objective of a test.

## 4 LARGE-SCALE TESTING VS. SMALL-SCALE TESTING

Most accelerated tests are performed on relatively small test samples. The advantage being that the test equipment may be kept relatively small, light and inexpensive. Further advantages are that large acceleration factors can be achieved especially for UV radiation and heat. The disadvantages are that relatively few samples may be tested at the same time and that small homogeneous samples may give quite a different result than large more, complex ones. The most important shortcoming, however, is that components and assemblies may be too large to be tested in such equipment.

Conversely, the advantages of large equipment are that large samples may be tested and that they offer the possibility of testing more complex objects like components with several materials, including joints etc. Large samples of a building material may give stress/strain situations that will not be observed on smaller test specimens e.g. the compatibility of materials with different dimensional stability to moisture and temperature. The influence of the total design of a specimen including geometrical shape, joints etc. may be studied, and large samples give the opportunity for more or other tests of the materials/components after the artificial ageing. Also the possibility of testing large numbers of samples at the same time is of course an advantage. The disadvantage is that this type of equipment usually is heavy and expensive and that the size set limits to the location of the equipment. Also the problem of establishin equal climatic load throughout the whole test area may be seen as an disadvantage.

Basically it can be said that the two types – small and large – are good each for their purpose. In other words they complement each other. The small equipment is best suited for smaller samples of materials and components and the large equipment for larger samples of components and assemblies.

## 5 DEVELOPMENT OF LARGE-SCALE TEST EQUIPMENT

In 1965 an equipment for accelerated ageing of building materials and components was built at the Norwegian Building Research Institute (NBI). The main objectives were:

1. To be able to test objects larger than what was possible in the then commercially available Atlas Weather-O-meters and similar
2. To have a sufficiently large ageing factor.

The method was especially designed to study the degradation of vertically positioned materials and components in the building envelope when exposed to the climatic strains: UV radiation/heat, rain and frost. Later other and bigger large-scale equipment using the same principles was built. At NBI a test equipment was also developed for studying the impact of the same climatic strains on horizontal building materials, components and assemblies.

In all cases the idea is that it should be possible to test whole building components e.g. entire windows or entire facade components. In general the test specimens are mounted in a manner that best simulates the in-use conditions and in a way that they are not exposed to other strains than those experienced in practice. Test specimens exposed simultaneously must be unable to influence each other.

The results of the test are given as any change in appearance of the specimens during the test, any signs of degradation e.g. cracks, loss of gloss, de-lamination etc. together with information on when the changes occurred, how big they are etc. The result of an accelerated testing is often a change in performance properties. Such properties may be tested before, during and after the artificial ageing. To detect the changes in properties, suitable test methods are chosen dependent on the material or component under test. Destructive tests must of course be performed on different specimens, which means that test specimens must be removed from the ageing equipment during the ageing test.

In the following a brief description of the various types of equipment is given together with some examples of their use.

### 5.1 Climate carousel – “The Four Seasons”

For many years the testing procedure of the first large-scale test equipment in Norway and Denmark was described as a common Norwegian/Danish standard. However after more than 30 years use the time was found to be ripe for revision of the test method which is currently being processed to become a Nordtest Method (NT Build Method 495).

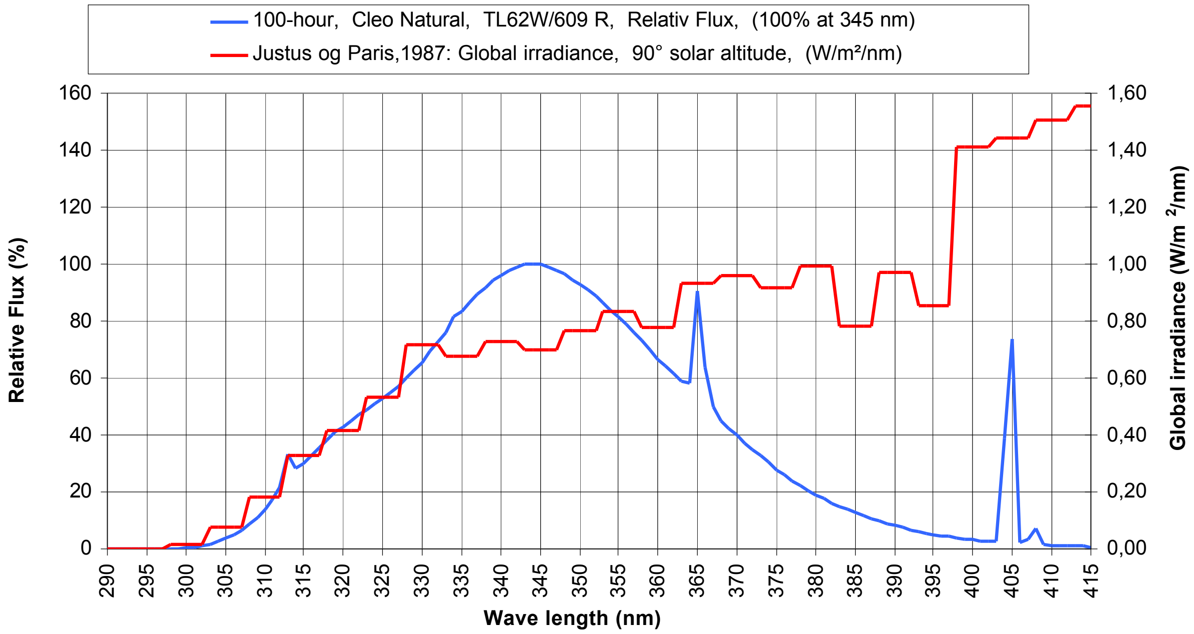
The revision primarily focused on improving the spectral distribution of the UV lamps used, give a better definition of the black panel temperature etc. The test method describes the procedure including the exposure. The test method is, however, rather open to changes in the size and duration of exposure to the ageing factors, as it is envisaged that the individual factors will need to be adapted depending on the tested materials, components and assemblies.

#### 5.1.1 UV light

The challenge was not to find a UV light source capable of providing a great intensity as regards UV-light but to find a find a suitable source that had a spectral distribution comparable to that of sunlight as well as the necessary physical properties to me it suitable for use in equipment with a large exposure area. Moreover it was found to be crucial in order to avoid degradation mechanisms not occurring under normal use, no radiance should be found apart from what is found in natural sunlight.

Selection of a suitable light source was done according to the information in ASTM G 53 – 96 (non-metallic materials) and in EN/ISO 4892-3, 1999 (Fluorescent UV lamps). Further the spectral distribution of the potential light sources was compared with the spectral distribution of modelled data for atmospheric conditions in CIE 85-1989, Table 7 (1) found by Justus and

Paris (1987). Philips Cleo Natural 59 W-R, which is a fluorescent UV tube, has been found suitable for the purpose giving a good fit to the spectral distribution in the UV band up to about 365 nm, cf. Figure 1.




5.1.2 The test equipment

The equipment consists of 4 test frames shaped as boxes with a closed bottom normally insulated behind with 50 mm mineral wool. The test samples can be mounted in the openings with hooks, screws or similar. The central unit rotates 90° at fixed time intervals so that each test frame is in turn exposed to the four different exposure climates, see Fig. 1. At present it is, depending on the used equipment, possible to test specimens with a maximum size from 0.9 x 0.9 m to 1.5 x 2.5 m. When not exposed, the test frames are facing the laboratory allowing inspection and measurements of the test samples.

From this follows that the exposure takes place in 3 climatic chambers plus the ambient climate. When rotating the carousel, the test specimens are in turn exposed to the following strains: A: UV/IR radiation, B: Water, C: Frost and D: Laboratory climate.

The UV lamps are normally placed 500 ± 50 mm from the test specimen but the distance may be reduced to 350 mm without harassing the uniformity of the UV light. The lamps are systematically replaced in turn in a way that the irradiation on the test specimens varies as little as possible during the test period. The black panel temperature shall rise to its designated temperature in the course of 45 minutes from the beginning of the exposure to UV light and heat radiation. The black panel temperature is normally 63 ± 5°C but, if required, the temperature may be chosen to be 35 ± 5°C, 50 ± 5°C or 75 ± 5 °C instead, cf. ISO 4892. The temperature is controlled by means of IR halogen lamps. The black panel is an un-insulated, non-gloss, black, flat aluminium panel with nominal dimensions: 100 mm long, 75 mm wide, and 2 ± 0,5 mm thickness with a suitable black coating which absorbs more than 90% of all incident radiation up to 2500 nm, for example black anodized 6063 aluminium.

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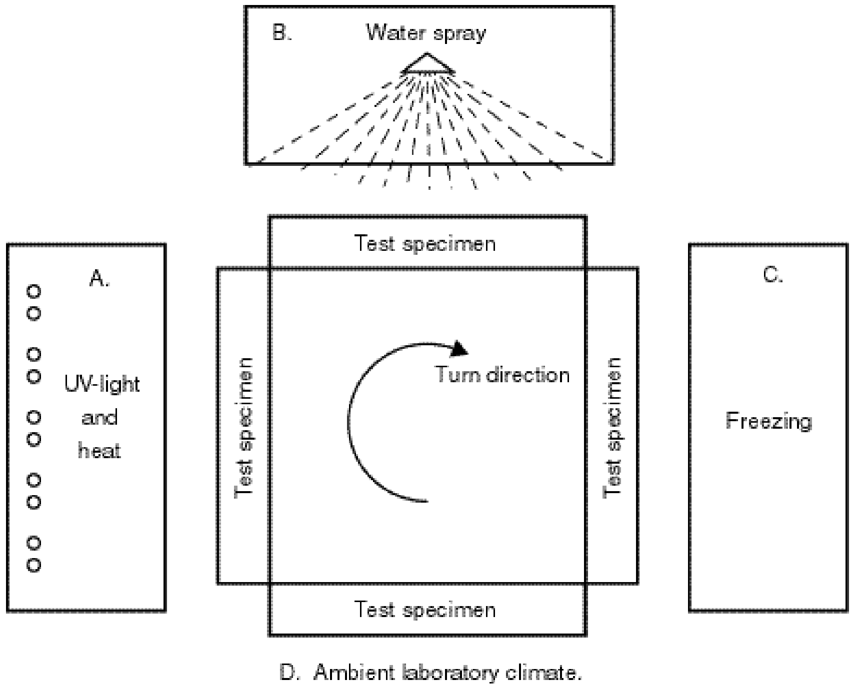



**Figure 2: Sectional view of climate carousel – "The Four Seasons"**

The UV tubes as well as the IR lamps may be regulated in order to control the UV irradiance and the surface temperature of the test specimen, if required.

Wetting of the specimen is done with a spray of de-mineralised water normally with 15 ± 2 l/($m^2$ h). If required, various spraying conditions may be used regarding for the amount of water and the spraying period.

Cooling and freezing air temperatures of –20 ± 5 °C are obtained, e.g. by supplying cooled air to the climate chamber. Other specified air temperatures may be used, if registered and reported. The air temperature is measured with a white panel sensor, similar in construction to that of the black panel temperature, and placed in the proximity of the specimen centre.

After freezing the specimens are exposed to thawing at ambient laboratory climate, i.e. 23 ± 5°C, 50 ± 10 % RH. During thawing the specimens may be inspected, rearranged and/or changed.

The time interval in each position is chosen dependent on the material/components under exposure and should be at least 1 hour. The limited possibilities of changing the ratio between the ageing factors are to change the UV and/or IR radiation, and/or by shortening the duration of the water spray.

5.1.3 Example of the use - Testing of renovation systems and paint for windows

For reasons of sustainability great efforts have been made to maintain or re-use old wooden windows, making windows from old wood and treating wooden windows with environment-friendly paint without wood preservatives. For this reason more investigations have been performed on windows including inspections in situ as well as artificial ageing.

Artificial ageing is performed on entire windows i.e. frames as well as sashes. In this way it has been possible to assess the behaviour of alternative constructions, including the influence of factors such as the wood quality, the geometrical shape of the sashes and frames, the corner joints, the putty material on the durability.

Assessments are performed during and after exposure and are based on signs of deterioration indicated by spalling, cracking, blistering etc. In some cases also the moisture uptake during ageing has been used as an indicator of the potential risk of mould growth and dry rot – which cannot be assessed by artificial ageing. For some windows, e.g. renovated windows or windows manufactured from old wood, the artificial weathering has further been supplemented with a performance test of air and water tightness of the window.

It appears form the test results that the quality and pre-treatment of the substrate/wood is crucial to achieve a good result. Quite surprisingly it was found that unpainted windows performed quite well which is believed to be due to their good possibilities of drying which shorten the periods with dangerously high humidity.

It was found that linseed oil paint is able to perform pretty well. However, modern linseed oil paints no longer contains lead etc. like earlier used types and therefore it may be necessary to add small amounts of fungicide to the paint for use on the most exposed parts of the windows. Besides the painting process is quite complicated.

**Figure 3: Example of accelerated testing of different paint systems used on window sashes.**

5.1.4 Example of the use - Ageing of fibre-reinforced cement claddings

Another example is testing of 8 mm fibre-reinforced cement claddings for exterior use. This has been a very important subject in order to find substitutes for asbestos fibres. The panels were cut into test pieces each measuring 600 mm x 600 mm. Pictures were taken before and during the test period, cf. Fig. 4 showing two of the panels with different fibre materials after 48 weeks in the climate carousel being rotated every hour. As can be seen one is nearly unaffected while the other is severely deteriorated. The ageing was supplemented with test of mechanical properties in order to study the influence of composition, including the fibre material, on the durability especially as regards the mechanical properties.

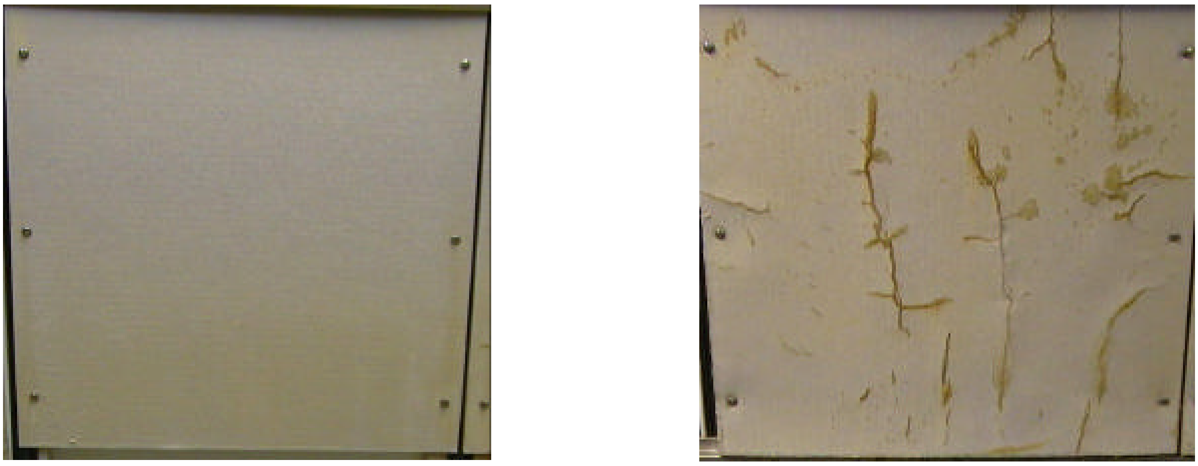


**Figure 4: 8 mm fibre-reinforced cement claddings for exterior use after 48 weeks in the climate carousel**

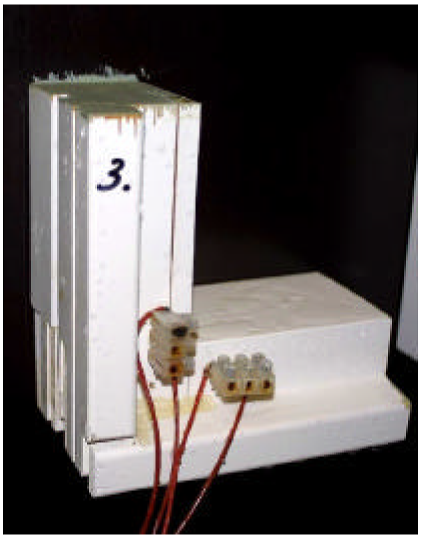



**Figure 5: Testing of window corner supplemented with measurement of moisture content in the wood during the test.**

**Figure 6: Example of accelerated test of sealant materials used between windows and wall.**

### 5.2 NEW Equipment For ageing of vertical building components AT DBUR

Originally the main purpose with this test equipment was to improve the capacity for accelerated ageing at DBUR and to test larger specimens than it was possible to do in the earlier equipment, whereas there was no intention of changing the exposure compared with the climate carousel. In practice, however, it turned out to be somewhat different from the carousel as it was decided to use other types of lamps and another concept for the control.

#### 5.2.1 The test equipment

The test equipment consists of a test frame with an area of approximately 10 $m^2$ (3.2 x 3.0 m). The test specimen is exposed in only 1 climatic chamber where it is in turn exposed to the following strains: 1. UV light and heat radiation, 2. Water, 3. Frost, and 4. Thawing at room temperature.

The same considerations as for the climate carousel, were made regarding selection of a suitable UV source were done. In this equipment, however, the UV radiation comes from UV lamps where for example Osram Ultratech 400 and Philips HPA 400/30S have been found suitable for the purpose. The lamps are equipped with filters to get rid of UV-C radiation and measurements show a satisfactory spectral distribution. Control of black panel temperature and the wetting and cooling/freezing is done in the same way as with the climate carousel. Thawing and exposure to ambient laboratory air climate, i.e. 23 ± 5°C, 50 ± 10 % RH is done by ventilating the climate chamber with laboratory air.

The time interval in each position is chosen dependent on the material/components under exposure and should be at least one hour. The ratio between the ageing factors is controlled freely i.e. there are no restrictions to the exposure pattern for this equipment. If desired, the exposure may be chosen in accordance with the climate carousel.

#### 5.2.2 Examples of the use - Ageing of a wooden facade

An example on the use of the test equipment is ageing of an entire facade element with exterior wood cladding including joints, windows, flashings etc., built up to copy the facades of an existing building. For this particular ageing test it was anticipated that the primary degradation factors were moisture changes in the materials, including joints, combined with the UV light and heat.

The exposure cycle was selected with prolonged exposure to UV light compared with the wetting (because humidity is to a large extent forced into the test specimen by the heat) and the action of frost was left out as it was considered of minor influence.

After 6 months testing considerable deterioration was found including cracks in the wood, curving of the panels, opening of joints between boards in the cladding, pull-out (5-15 mm) or pull-through of nails and deterioration of the paint. The deterioration obtained in the ageing equipment was compared with the deterioration of the facade of the actual building. The comparison showed a high degree of agreement between the natural and the artificial ageing. As the artificial ageing was 6 months and the natural ageing 10 years an acceleration factor of about 20 was found for this specific purpose.

**Figure 7: Example of full scale testing of a wooden facade including windows.**

### 5.3 Equipment for performing accelerated ageing tests on horizontal building components

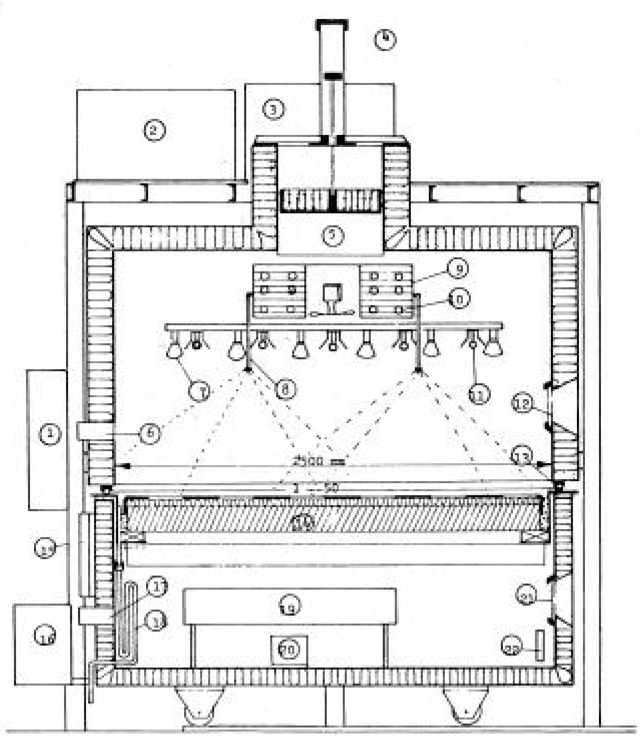
At NBI an equipment has been developed for studying the durability of materials and components placed in a horizontal position, mainly flat or low-pitched roofing.

#### 5.3.1 The test equipment

The test equipment consist of three main parts, see Fig. 8.

- An upper box for simulation of outdoor climatic strains.
- A lower box for simulation of indoor climate.
- A steel frame for roof assemblies of 2.4 m x 2.4 m.

In the test specimen is in the upper box exposed to the same strains as in the climate carousel. The duration of each parameter can be varied independently. When performing an accelerated ageing test, an appropriate indoor climate is chosen and contro lled in the lower box. Movements in the test material/components may be achieved by applying pulsating air pressure.



UPPER BOX

*5.3.1.a.1.1 LOWER BOX*

1 Temperature control and sequence timer
2 Cooling device
3 Instrument panel
4 Pneumatic cylinder
5 Air pressure pulsator
6 Air suction
7 IR lamps

8 Water spray
9 Air cooler
10 Heating elements
11 UV tubes
12 Inspection window
13 Inflatable rubber hose

14 Test material/components
15 Instrument panel
16 Cooling apparatus
17 Air pressure inlet
18 Water outlet
19 Air cooler with heating elements
20 Humidifier
21 Inspection window
22 Hygrostat

**Figure 8: Test equipment for horizontally placed materials and components**

The test equipment may also be used for wind uplift tests at temperatures between –30°C and +90°C. It is very convenient to be able to control the temperature during a wind uplift test since the heaviest wind loads in the Nordic countries normally are expected between –5 °C and +15 °C. These tests are performed according to the Nordtest method, NT BUILD 307 “Roof coverings: wind load resistance”.

### 5.3.2 Examples of use Roofing products

Roofing products are tested in the ageing equipment mainly to assess the durability of the products and to find a basis for calculating the design capacities for mechanical fasteners in roofing felts in connection with technical approvals. The equipment complies with the requirements stipulated in prEN 1297:2001 “Flexible sheets for waterproofing – Bitumen, plastic and rubber sheets for roof waterproofing – Method of artificial ageing by long term exposure to the combination of UV radiation, elevated temperature and water”. In addition the equipment also may take into consideration both frost and movements in the sheet to be tested.

### 5.3.3 Liquid applied membranes for maintenance of old roll roofing

Liquid applied roofing membranes (LAM) were applied to old roofing felt and tested immediately after curing for 28 days as well as after artificial ageing for 16 weeks in a horizontal position.

The same materials have been exposed to natural ageing at different locations in the Nordic countries for periods from 6 months to 12 years. The visual inspections after natural and artificial ageing both showed the same type of defects. Both also showed that moisture is damaging to the liquid applied membranes.

Testing of mechanical properties showed that the old roofing felts gained both strength and flexibility after being coated with LAM. Meanwhile, the gained properties are greatly reduced after 16 weeks of artificial ageing.

The conclusion from this investigation was that when properly applied to dry, old roofing felt LAM may serve as a temporary alternative to re-roofing.

**Table 1:Tensile properties and cold-bending temperature for liquid applied membranes.**

| Material | Tensile properties (EN 12311-2 A) | | | | Cold bending |
|---|---|---|---|---|---|
| | Force Longitudinal | Force Transverse | Elongation Longitudinal | Elongation Transverse | (EN 1109) |
| | N/50 mm | N/50 mm | % | % | °C |
| Old roofing | 450 | 205 | 2.8 | 2.4 | + 10 |
| L1 asphalt | 585 | 336 | 3.4 | 4.6 | + 10 |
| L1 aged* | 633 | 431 | 1.9 | 2.8 | > + 20 |
| L2 acrylic | 418 | 316 | 2.3 | 4.4 | + 5 |
| L2 aged | 641 | 371 | 1.9 | 1.7 | + 20 |
| L3 asphalt emulsion | 536 | 301 | 3.9 | 6.5 | - 5 |
| L3 aged | 630 | 476 | 2.0 | 4.0 | + 5 |

L# = Old roofing felt with a layer of a liquid applied membrane (LAM) number #

*Artificial ageing according.

### 5.3.4 Example: Roofing membrane of ethylene-co-polymer/bitumen

A roofing membrane of ethylene-co-polymer/bitumen was collected from a roof in Jönköping, Sweden. The membrane had been on the roof unprotected for 4 years, lying on asphalt felt. This membrane was tested for tensile properties and tear – nail shank according to test methods close to respectively EN 12311-2 method A and EN 12310. The new membrane was tested after exposure to artificial weathering for 72 weeks (NS 8140/DS 1127), heat ageing at 70°C for 24 weeks (EN 1296) and ageing in hot water at 60°C for 8 weeks (NS 3531).

Roughly the test results showed that no significant changes was found in the membrane exposed to heat or hot water, whereas the artificial weathering causes a slight reduction in the measured properties.

The conclusion is that the product has very good weather resistance, but that some migration between asphalt felt and the membrane is causing deterioration of the membrane during practical use on the specific roof in Jönköping.

**Table 2: Tensile properties and tear resistance for roofing membrane of ethylene-co-polymer/bitumen.**

| Material Treatment | Tensile properties/St.d. (EN 12311-2 A Mod*.) Force Longitudinal | Force Transverse | Elongation Longitudinal | Elongation Transverse | Tear – nail shank/St.d. (EN 12310 Mod*.) Longitudinal | Transverse |
|---|---|---|---|---|---|---|
| | N/50 mm | N/50 mm | % | % | N | N |
| New material | 1020/130 | 876** | 408/72 | 688** | 299/6 | 285/10 |
| Natural weathering 4 years | 500/25 | 407/5 | 256/14 | 400/14 | 181/5 | 220/5 |
| Artificial weathering 72 weeks*** | 999/92 | 699/59 | 430/53 | 616/39 | 200/5 | 258/10 |
| Heat ageing 24 weeks at 70 °C | 1040/100 | 739/126 | 452/26 | 627/140 | 216/2 | 268/9 |
| Ageing in hot water 8 weeks at 60 °C | 1030/10 | 766/6 | 465/11 | 703/20 | 205/11 | 267/4 |

For new material 5 specimens are tested. For artificial aged/weathered material 3 specimens are tested

* Modification EN 12311: 40 mm/min. Modification EN12310: 50 mm specimens/3 mm pre-cut hole for shank.

** Only one specimen is tested.

*** Planned for 48 weeks, but prolonged as no significant changes was found after 24 and 36 weeks compared with new material.

Artificial weathering in accordance with NS 8140/DS 1127

Heat ageing in accordance with to EN 1296

Ageing in hot water in accordance with to NS 3531

## 6 CONCLUSION

Large-scale equipment for accelerated ageing is considered to be a valuable tool not only to accelerate the ageing process of single building materials but also to provide insight in how different materials or components interact with each other under the influence of different exposure factors. For example the compatibility of materials with different moisture and temperature deformation properties and the influence of the total design of a specimen including geometrical shape, joints etc. may be studied.

To the authors experience the results achieved show good correlation with the deterioration experienced in practice. This means that if you find a ranking e.g. for different paint systems after artificial weathering, you should find the same ranking after natural weathering independent of geographical location and situation - on condition of course that the paint systems are used in situations that are sufficiently comparable.

In some (few) cases it may be possible to find an acceleration factor for specific uses of materials/components.

## 7 ACKNOWLEDGMENTS

The project on revising the test method for the climate carousel was sponsored by Nordtest.

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# DURABILITY OF FAÇADES WITH CERAMIC COVERINGS – WHY THEY FAIL

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**Summary: This work is based on the most common pathological descriptions which occur in façades with ceramic coverings, to develop a method for analyzing which allows causes to be determined and courses of action to be proposed which, once implemented in the project, execution and maintenance, can avoid such problems from appearing.**

**The article discusses the concept of durability and service life, as well as the factors which affect the parameters used when a designer defines the projected design life, along with the procedures required to for this to become the real service life of the ceramic covering sub-system.**

**The work in this study was carried out in São Paulo - Brazil, where a residential building was studied where the ceramic coverings presented tile loosening, and where we attempted to apply the aforementioned concepts. The study describes preventive and corrective actions for the pathological manifestations observed.**

**Keywords: Durability, Fail, Facade, Covering, Ceramic tiles**

## 1 INTRODUCTION

The main reason that buildings in Brazilian cities have had ceramic coverings for decades is mainly their high level of resistance in varying climatic conditions, and the material does not usually present high levels of wear and tear over time.

The fact that ceramic coverings last longer is measure by Shohet & Laufer (1996). When they compare this type of covering with others, such as cement-based mortar, synthetic mortar and stone, a clear superiority of ceramic coverings can be seen when compared with mortar coverings, in accordance with Table 1.

**Table 1: Estimates of service life of various types of external coverings in different environments (Shohet & Laufer, 1996)**

| Environment category | Cementitious stucco | Synthetic finished stucco | Ceramic covering | Stone |
|---|---|---|---|---|
| Non-corrosive | 10-15 | 12-15 | Over 15 | over 25 |
| Corrosive | 5 years | 8-12 | 10-15 | over 25 |

This capacity to remain unaltered over e period of time, associated to other capacities listed by Medeiros (1999), such as: ease of composing geometric patterns, easy cleaning, good resistance to humidity, general satisfactory performance and low maintenance costs, are the main reasons for using this type of covering on buildings, especially at sea-side locations.

Ceramic façade coverings are taken to be: a monolithic set of layers (including the plaster layer) adhering to the building support base (brickwork or structure) whose outer covering is composed of ceramic tiles, laid and grouted with mortar or adhesive material", (Medeiros, 1999).

Understanding the various properties of each of these components and the behavioral interfaces between them is fundamental for avoiding the appearance of pathological manifestations, faults, which can occur at a very early stage, in accordance with an analysis by Campante & Sabbatini (2000), which saw that 50.9 % of the buildings studied showed tile loosening on the façade within 5 years of use.

## 2 PATHOLOGICAL MANIFESTATIONS IN FAÇADE CERAMIC COVERINGS (FCC)

Pathological manifestations in FCCs is understood to mean situations where, at a certain moment in the working life the product fails to present the expected performance, in other words, no longer complies with the functions for which it was designed and no longer meets the needs of the users.

These functions can be summarized as: protection of the building's sealing elements, help the sealants to fulfill their functions (acoustic and thermal isolation, proof against water and gases, etc) providing a uniform finish to the sealing elements as a final finish, providing aesthetic functions, for economic value and use of the building.

This means that the main pathological manifestations which occur in FCCs can be classified in three categories: efflorescence; cracks and fissures; and loosening.

Efflorescence is taken to mean the formation of crystalline deposits on a surface due to physical, chemical and physical-chemical actions. Normally, it is caused by the movement of water through the porosity of the FCC payers, transporting alkaline metal salts (sodium and potassium) and earthy alkalis (calcium and magnesium), which are soluble or partially soluble in water, until they appear on the surface after the water evaporates.

The cracks and fissures are the loss of integrity in ceramic component's surface, which can also cause loosening of this component from the ceramic covering sub-system. Cracks are defined as a total rupture of the ceramic body in two or more parts after fixing, whose size is above 0.5 mm. Fissures are opening of 0.01 mm to 0.05 mm of the glazed finish on the ceramic tile. (Sabbatini & Barros, 1990).

The loosening of tiles means the process of failure between the ceramic tiles and the fixing layer, to the latter and the substrate, characterized by the separation of sections of the covering at points or in general, caused by the appearance of tensions which are greater than the capacity for resistance of the adhesive links.

Considering the consequences (economic and personal risk), this could be considered the most serious of FCC. This is because of the effects of this type of problem: a ceramic tile weighing 250 g falling from the 10$^{th}$ floor of a building has the same destructive power as a projectile from a firearm when it reaches the ground (Tan *et al*. 1994).

most of the pathological manifestations which occur in FFCs can be attributed to several factors which occur at the same time. This means we need to carry out a hierarchical analysis starting by observing the defect, then discovering the immediate cause, then its nature, or secondary cause, and arrive at the origin of the problem, or primary cause, as seen in figure 1.

| | |
|---|---|
| Problem perceived | DEFECT or PATHOLOGICAL MANIFESTATION |
| 1st level of observation | IMMEDIATE CAUSE |
| 2nd level of observation | NATURE |
| 3rd level of observation | ORIGIN |

**Figure 1: Hierarchical analysis of pathological manifestations in FCCs**

Applying this line of thought to analysis of tile loosening problems starts with the observation pf the **defect**, followed by a search for the first cause, or **immediate cause,** in this case the fault in adherence between the ceramic tile and the fixing mortar. Then we seek the secondary cause, the **nature** of the defect, which could be, in this hypothetical case, the application of the tile onto the mortar too long after the mortar has been applied. The we should seek the cause of this problem, and reach the **origin** of the defect, in other words, deficiencies in the execution of the work, which could be caused by lack of training and/or control (or even an error in the specification of the fixing mortar).

According to Cheong (1992) the analysis of the loosening could lead to four distinct natures: adhesion failure between the ceramic tile and the fixing mortar; lack of adhesion between the fixing mortar and the substrate; faults in the substrate layers; and a hollow sound in the ceramic tiles when hit.

For Sabbatini & Barros (1990) the main factors associated to the origin of tile loosening are: deformation in the bases (brickwork/structure) due to settling of the building after occupation, flow in the concrete structure, which is not seen immediately and hydrothermal variations; faults in the control joints; inadequate mortar for plastering, laying and grouting and deficient base preparation.

One of the main causes of tile loosening is the change from the mechanical resistance offered by the various layers to the mechanical force actually applied. These forces come from movement in the FCCs, caused by movement in the building or changes to the conditions to which they are exposed.

For Medeiros (1999), there is also a fourth level. defined by him as the "Genesis" of the problem, which has two parts:

1. A lack of projects which account for performance parameters and which consider the requirements of the production process.
2. A lack of technological understanding of covering production by the entire production chain, starting with the engineers and architects and going down to the tile layers.

## 3 THE CONCEPT OF DURABILITY AND APPLICATION TO FCCS

In the FCC sub-system, quantification of durability takes on fundamental importance for two reasons: firstly because the use of this data allows us to define the economic viability of using certain types of component and materials. The second reason is allowing for comparison of the presented durability with other alternatives, so that we can choose the best one for the conditions seen.

Establishing durability of a subsystem means first understanding the deterioration mechanisms associated to each of the FCC components, and from there suing the same concept for the subsystem, analyzing the degradation mechanisms. In other words:

**Deterioration** is a term associated with the **components** of the subsystem, and

**Degradation** is a term directly linked to the **subsystem** as a whole

For CIB (1983), durability must be studied from four different aspects:

I. material used, which are classified on a scale from perishable to non-perishable, according to the level of deterioration during use;

II. project, which will also be analyzed using a scale from bad to excellent, according to factors considered for the material in question;

III. conditions of usage, which must be classified from severe to weak;

IV. maintenance, which must be evaluated on a scale of frequent to non-existent.

These aspects account for the subsystem material, and the subsystem as a whole. This is explained by the fact that in determining durability of a subsystem such as FCCs, we must understand the interface mechanisms between the various components, which is still a relative unexplored area.

## 4 CONCEPT OF WORKING LIFE AND APPLICATION TO FCCS

Service life, for FCCs , can be defined as:

*The period of time where you can maintain the properties that allow it to carry out the functions for which it was designed, taking into account the remaining time of use and the estimated costs of maintenance and substitution.*

When talking of FCC Service life, two parameters must be defined: design life (DL) and service life (SL).

Design life (BSI, 1992) can be defined as the pre-established value given by the designer which should be aimed fro during usage. This will be the basis for choosing materials, techniques of subsystem execution, maintenance actions and costs to be taken on by the user.

There are a series of conditions for DL, among which the life cycle of the building, which will define the period of time required for the subsystem to maintain its original characteristics. This means, in general terms, that FCC DL must be specified as the same as that for building substrates and sealing systems. However, DL of the subsystem must not be confused with that of its components. In Table 2 some criteria are listed which defined the different values for the subsystem components.

**Table 2: DL estimates for FCC components**

| Subsystem component | Classification of importance in the subsystem | DL Class |
|---|---|---|
| Fixing layer | A1 | 1 |
| Ceramic Tile | A2 | 1 |
| Grouting | B | 2 |
| Expansion Joints | C | 3 |

The table above classified the diverse components in accordance with the effects of their faults. This means that a component classified as A1 would cause subsequent failures after its own. What we are trying to establish is a relationship between the various components; a failure in the fixing mortar would cause the collapse of the entire subsystem, causing the ceramic tiles to fall off. For this reason, its DL must be classified as 1, in other words the same value as the subsystem as a whole.

In order to make it easier for façade designers to take decision as to how to establish the different DLs for the FCC components, a ´proposal is summarized in Table 3. This means that a durability table can be provided for the FCCs, which can be used to choose the period of time that the subsystem must retain its initial characteristics as a result of the variables: Production project, Subsystem component materials and Procedures for carrying out the subsystem;

**Table 3: Durability estimates for FCCs based on DLs**

| **Quality levels of variables** | **Levels of durability** | | | | |
|---|---|---|---|---|---|
| | A | B | C | D | E |
| PPP | 10 | 7 | 6 | 4 | 2 |
| MAT | 8 | 6 | 4 | 3 | 1 |
| EXE | 12 | 8 | 7 | 6 | 2 |

Where: PPP: Design for Production

MAT: Material of the subsystem components

EXE: Procedures for subsystem execution

Level A: 25 to 30 points ⇒ over de 25 years

Level B: 20 to 25 points ⇒ 20 years

Level C: 15 to 20 points ⇒ 15 years

Level D: 10 to 15 points ⇒ 10 years

Level E: 5 to 10 points ⇒ less than 5 years

The points described in this table are obtained from an analysis of each of the three basic factors listed in the table. As a result of the quality level of each of these factors, we can define how long the FCC should be expected to last.

After defining the level of influence of each of the variables, each one is given a weight, which are defined in Tables 4, 5 and 6, as follows.

**Table 4: Maximum weight attributed to the factors associated to the PPP variable in defining the FFC subsystem durability**

| INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT |
|---|---|---|---|
| **Analyses and definitions** | **5** | **Specification and details** | **5** |
| Physical feasibility | 0,2 | Define base and substrate preparation | 1,0 |
| Economic feasibility | 0,2 | Define substrate joins and reinforcement | 1,0 |
| Financial feasibility | 0,2 | Define construction details | 1,0 |
| Exposure conditions | 1,1 | Define construction methods | 1,0 |
| Façade architecture | 1,1 | Define control criteria | 1,0 |
| Structural deformability | 1,1 | | |
| Characteristics of external brickwork | 1,1 | | |

**Table 5: MAXIMUM WEIGHT attributed to the factors associated to the MAT variable in defining the durability of the FFC subsystem**

| INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT |
|---|---|---|---|---|---|---|---|
| **Study of tile type** | **2** | **Material selection for fixing layer** | **2** | **Material selection for grouting layer** | **2** | **Material selection for expansion joints** | **2** |
| Historic problems | 0,1 | Fixing/grouting mortar properties | 0,5 | Deformation capacity | 0,4 | Size of expected FCC movement | 0,3 |
| Direct/indirect costs | 0,1 | Application surface characteristics | 0,2 | Lateral adherence of ceramic tiles | 0,3 | Locations of tension concentration in the FCC and base | 0,3 |
| Wall backing surface treatment | 0,3 | Forecast structural behavior of FCC | 0,5 | Workability | 0,4 | Tile modularity | 0,3 |
| Tile weight | 0,3 | Environmental conditions | 0,3 | Drying retraction | 0,4 | Position/thickness of laying joints | 0,2 |
| Porosity of backing wall | 0,3 | Tile position | 0,2 | Esthetic requirements | 0,1 | FCC and base material characteristics | 0,3 |
| Tile thickness and surface | 0,3 | Relative façade position | 0,3 | Resistance to deterioration | 0,4 | Sealing properties | 0,3 |
| Tile color | 0,3 | | | | | Geometry / dimensions / quantity / expansion joint position | 0,3 |
| Tile properties | 0,3 | | | | | | |

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**Table 6: MAXIMUM WEIGHT attributed to the factors associated with the EXE variable when defining the durability of the FCC subsystem**

| INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT | INFLUENCING FACTORS | MAXIMUM WEIGHT |
|---|---|---|---|---|---|---|---|
| **Service program** | **1** | **Labor definition** | **4** | **Technical definition of application** | **5** | **Project control** | **2** |
| Logistics | 0,1 | Team sizes | 0,3 | Procedure for material preparation/handling | 0,5 | Define process control stations | 0,7 |
| Tools / equipment | 0,3 | Procedure preparation | 1,5 | Marking/leveling/expansion joints/rows techniques | 0,5 | Define criteria for receiving services | 0,7 |
| Installation / movement of equipaments | 0,3 | Training manual preparation | 1,5 | Techniques for adjusting covering dimensions to project modulation | 0,5 | Project process Feedback | 0,6 |
| Supply/handling of material | 0,1 | Labor qualification | 0,7 | Techniques for applying fixing mortar | 0,5 | | |
| Production organization | 0,2 | | | Techniques for placement of ceramic tiles | 0,5 | | |
| | | | | Techniques for using manual tools | 0,5 | | |
| | | | | Techniques for applying grouting mortar | 0,5 | | |
| | | | | Techniques of executing joints | 0,5 | | |
| | | | | Techniques for executing expansion joints | 0,5 | | |
| | | | | Techniques for cleaning after application | 0,5 | | |

The service life is defined by BSI (1992) as the period of time during which a component or construction maintains its characteristics, exceeding the users' requirements as to operation, maintenance and repair, in other Word, the period during which the subsystem **really** maintains its properties up to the minimum acceptable level to meet users' expectations

The SL of a subsystem is limited by what ISO (1997) called the deterioration to which components that cannot be substituted are subject. Deterioration itself may not require substitution of components that can be substituted, however, in some cases, substitution may be so expensive that it may affect the SL of the subsystem as a whole. This is often the case with FFCs.

## 5 CASE STUDY

A high quality residential building located in a middle to high class region of the city of São Paulo, with 20 floors and a penthouse. This case study began with a request from the construction company for an evaluation the cause of tile loosening which was occurring mainly in the front and rear façades while it was being handed over to the owners.

Diagnosis began with an on site visit, which showed the location of the building. The main problems with loose tiles were occurring on the northeast (frontal) and southwest (rear) façades, with the latter most affected, as can be seen in figure 2.

**Figure 2: View of the façade affected by loose tiles**

During the inspection, high winds were detected around the area due to the relatively isolated position of the building on a hill. The most affected façade was also subject to high quantities of direct sunlight in the afternoons. it was also noted that no type of expansion joint was used, either vertical or horizontal, as can be seen in figure 3. There is also a lot of cracking in the grouting mortar, as shown in figure 4. Another fact observed was that many ceramic tiles which had fallen off showed a lack of contact with fixing mortar on the contact side.

**Figure 3: Evidence of lack of expansion joints on the right rear façade.**

**Figure 4: Evidence of cracks in the grouting between tiles.**

Interviews with the people responsible for the building did not provide enough information on the stages of execution and material used to apply the ceramic tiles, as the construction company does not exist any more. We could only determine that the tiles started to loosen and fall soon after the building had been delivered.

Even without access to all of the information, we attempted to define the **Nature** and **Origin** of the problem from the interviews and inspection results. From the point that the problem has been ascertained, we reach the 1$^{st}$ level of observation: the **Immediate Cause** of the problem, the lack of adherence between the tile and the fixing mortar shown by the lack of filling on the tile contact surface.

Moving on the next stage, determining the **Nature** of the problem, the following factors were found:

- the tiles were applied outside the appropriate period of mortar application;
- the tiles were applied without using the double fixing principle;
- the tiles were applied to a surface subject to severe weather conditions, such as: high levels of sun lights, high winds, and no adequate protection;
- base deformations;
- ceramic tile humidity expansion;
- retraction during drying of the fixing mortar;
- fatigue in the links between the fixing mortar and the ceramic tiles due to thermal shocks.

Once we had analyzed the **Nature** of the problem, we studied its **Origins:**

- inadequate choice of fixing mortar, indicated fro internal use and used o the façade;
- use of plaster and grouting mortar with inappropriate properties for the conditions (high level of retraction during drying and low capacity to absorb deformations);
- lack of expansion joints which could dissipate tensions to which the FCC was subjected;
- use of inadequate ceramic tiles, with high levels of absorption which required mortar with high levels of water retention; and
- use of inadequate fixing techniques for the type of tile used, with ceramic tiles with swallow-tail studs were applied using the final layer technique, which did not provide adequate filling of the tile contact surface.
- deficiencies (or lack of) execution procedures;
- lack of tile application training;
- deficiencies (or lack of) control criteria for carrying out services and accepting works;
- project deficiencies in the FCC subsystem, which did not allow for correct quantification of deformations to which the base would be subjected;
- high grouting mortar permeability which allowed d for tile humidity and humidity expansion;

Even reaching a series of diagnostic hypotheses regarding the problem, it was still not possible to determine the probable cause. We still had not examined the possibility of humidity expansion of the ceramic tiles above that permitted by the standards. Therefore, we turned to a laboratory to test the tiles, and the result was outside the range permitted by the NBR 13818 standard (ABNT, 1997): 0,.3 mm/m, against 0.6 mm/m. This would not recommend the use of this component for external purposes.

## 6 FINAL CONSIDERATIONS

This study has enabled us to discover the origins of the FCC loosening, and therefore establish how the project parameters, the building production procedures and the maintenance operations could influence the durability and service life of this subsystem, also leading to pathological manifestations. Within the project parameters, it is important to consider:

- Evaluation of the conditions which the façade will be exposed to, through a detailed study of the environment in which the building is placed, to determine the possibility of thermal shocks, freezing, levels of acid rain, wind, atmospheric pollution and others..
- Analysis of the façade structure: this stage should include variables such as geometry, forms, building type and level, surroundings, etc.
- Study of the ceramic tile typology: critical study which includes the parameters related to the choice of type of tile, including direct and indirect cost, history of problems, characteristics and properties of the ceramic tile, prior definition of the need for joints and types of fixing material.
- Evaluation of structure deformation: deformation over time, using variables such as: concrete deformation module, rigidity of the structure an its elements, slow deformation (flow), sequence and construction methods used in building the structure.
- Evaluation of external brickwork: technical analysis of the need to prepare the base and substrate for the FCCs, evaluating potential intrinsic movement of the sealing brickwork, mechanical resistance, absorption capacity and regions which could provoke tension in the FCC layers and influence the technical definition of the base preparation, plaster reinforcement and control joints.

The execution procedures should consider the most common errors which found during the field studies and correct them:

- Fixing to warm surfaces.
- Usage of incorrect fixing mortar.
- Fixing to contaminated surfaces.
- Spreading of fixing mortar over large areas.
- Dilution of fixing mortar to provide greater yield.
- Fixing over construction joints.
- Fixing on different materials without using joints.
- Fixing on dry fixing mortar.
- Fixing to damp surfaces.
- Fixing too fast.
- Fixing to inadequate bases.
- Incorrect spreading of mortar (without trenches or deformed trenches).
- Inadequate joint cleaning.
- Adulteration of products to increase profits.
- Substitution of the fixing mortar for another mortar, different from that specified for the project, generally inadequate for the climate and/or type of use.
- Omission of steps in fixing to gain time.
- Lack of works supervision
- Inadequate execution of the grouting and/or expansion joint layer.

As to the maintenance / use, this can be summarized in the following aspects:

- Use of unadulterated cleaning products.
- Clear maintenance procedures for inspection and possible exchange of subsystem elements which could be compromised, such as grouting and filing material in the expansion joints.

At the end of this study, we can reach two conclusions, the first of which is the importance of a construction log, or access to important documents which show the history of the building construction and the design of the FFC subsystem, such as: invoices, works logs, physical chronograms set and attained, service contracts, tests for materials received, “as built” projects, and others for diagnosing the problem.

The second is the recuperation of FFCs can be very costly. These costs include rental and installation of better rocker arms (to make services easier), the use of better quality material, low productivity because of small teams to better control the services carried out, difficult in finding ceramic tiles similar to those used during the works, labor suits, and others.

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# A Solution To Environmental Pressure And Housing Convenience

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**Summary**: **The growing demand for convenience in housing is increasing the pressure on the environment. To reduce this pressure it will be necessary to bring the technical life span and the life expectancy into harmony. Change of life span at the level of building component can provide a solution for this.**

**Within this scope, a research project is being conducted about the environmental pressure of buildings, in accordance with the residents' demands. With the technical knowledge of today, one is capable of producing products with a long life span. Improved materials and repairing techniques pay their tribute to this. However, as a result of the growing public purchasing power, building components are often replaced before it is a technical necessity, by which the environmental pressure will increase. To avoid this, the choice of materials has to be optimized. To make a well-considered choice in this, it has to be evaluated what will be the most efficient way to use materials throughout the entire life span of the building. In this assessment, the materials will be evaluated with LCA-techniques. Problem with most of these techniques is that life span is regarded as fixed specification, while life span is the aspect that is most influenced by variables. An example is the early replacement of building components, before their technical life span is over. Then questions arise such as how does this contribute to the overall environmental pressure produced by that building, and are there possibilities for the reuse of that component?**

**At the Eindhoven University of Technology (the Netherlands), a prototype building has been built according to the so-called IFD-technology (Industrial, Flexible and Demountable) that connects to the problem described above. This project is a collaboration between a housing association, building contractors and the university, where the building performances (now and in time) are being researched.**

**Especially flexibility and freedom of layout make it possible that changes in time are easily conducted without huge pressure on the environment. It results in a building according to the residents' wishes, but with limited environmental pressure for both the building components and the entire building.**

**Keywords. convenience, environmental burden, Life Cycle Assessment, life span, weighing model.**

## 1 INTRODUCTION - CONSTRUCTING

### 1.1 Constructing in general

Constructing still is an activity involving the employment of a lot of people, both during and after the construction period. It is a necessity to provide in sufficient living accommodation. Predictions (by the Dutch public housing department, Directoraat-Generaal van de volkshuisvesting Directie bestuurdienst, 2000) indicate that until the year 2004, 81.000 new buildings have to be constructed annually in the Netherlands. Throughout the years, constructing has changed from being a necessity to a way of providing facilities. It gives people a chance to distinguish themselves from other people, just by adding a personal touch to their home. In other words, constructing is an activity we cannot live without.

In Europe, there has been an enormous growth of the production of buildings since the end of World War II. At first it was to replace the destroyed buildings in war, but at a later stage also the habits of the people changed. In the 1960s it was normal that families had eight or nine children living under one roof. Sometimes people stayed to live there, even when they got married. Nowadays families are much smaller and children tend to move out much sooner. Health care has improved through the years, so that people are able to keep living on their own for a much longer period. Consequently, the demand for buildings keeps increasing, and now the world is full of people that all need homes. This demand for buildings will remain.

We do have a considerable stock of houses, but most of those are post war buildings that don't comply with the occupants' wishes. They are too small and do not have enough possibilities, for which reason they are mainly rented in the cheaper sector. These buildings are destined for major improvements to comply once more to the occupants' wishes. The most drastic measure is demolition. Other initiatives are IFD-today (see chapter 4) and the project 'The flexible breakthrough' (Hendriks, N.A., *et al*. 2002), (Vingerling, H., Hendriks, N.A. 2002).

### 1.2 Constructing and the environment

Environment-oriented thinking is growing in importance. At environmental conferences as in Kyoto (Japan) and more recently in Bonn (Germany), attempts were undertaken to come to joint agreements for reducing the damage to the environment. Contrary to the lack of results, those worldwide meetings are important because the subject of reducing pollution is again under discussion. An issue that is on the agenda every time is the reduction of the carbon dioxide equivalents. To reduce the green house effect, measurements have to be taken in different sectors, including the building sector. For the Netherlands, a reduction of the carbon dioxide level of six percent compared to the level in 1990 has to be achieved. The growth of the world population and the growth of prosperity make it necessary to reduce the environmental burden. At the same time this growth makes it harder to comply with the goals.

As mentioned in the introduction, constructing is an activity that always will exist. That is why the construction sector is one in which environmental profits have to be optimized, and the environmental burden has to be made as small as possible. One can make the difference between the materials used in the building (mainly concerning constructing and demolishing the building) and the use of the building. In case of using a building, energy-use is taken into account as well as material flows originated by maintenance and renovations.

### 1.3 Constructing and conveniences

The wishes of occupants are getting more and more important. In the early 1950s and 1960s, one had to be glad to have a building at all, nowadays every occupant has his own wishes and demands. People evaluate their houses by the conveniences they provide. This is a troublesome subject because what does convenience exactly mean? This will receive ample treatment in chapter 3.1.

Not only buildings, but also the interior and the use of building and interior is due for alterations. People are getting more and more wishes concerning the use of their homes. Examples of this can be found in visible components such as kitchen and bathroom. But also less visible components such as installation and constructions are subject to alterations. More and more, these components are being replaced by other, more modern and sophisticated, components. This increasingly happens before there is a real (technical) necessity to change the component.

Because of these wishes for convenience, replacement of components is carried out a lot. When reviewing the use of materials and putting a burden on the environment this will conflict with the replacement of components. At a time when the attention for the environment is increasing, a contradiction is forming: more resources are being used while the goal is to reduce the environmental burden. To take the environmental aspects into account, there has to be more understanding of the materials that are being used. A weighing model has to be found for the choices of materials. In this model, it isn't the calculations that are innovative, but the way they are used and the way the input is gathered that is the most important.

## 2 WEIGHING MODEL

The weighing model has to support the choice of materials or the choice of building concepts. The use of materials is one of the main issues in this model. During the life span of a building, alterations have to be made. These alterations will be taken into account when weighing the design. That way the result will be an overview of the complete life span and not just a random indication. Because the whole life cycle is taken into account, the basis on which the choices are made will be as accurate as possible. Different Life Cycles (LC's) can be related with the model: Assessments (LCA), Costing (LCC), Management (LCM). This survey is limited to the environmental aspects by means of Life Cycle Assessments (LCA). The weighing model has a huge number of variables, the most important of which are described below.

### 2.1 Building performance

Formerly the materials that were to be used in a building were exactly circumscribed. With the introduction of so called 'performance concepts' this became obsolete. Now products are circumscribed by the required performance, and the way the product fulfills this performance does not really matter. Of course it is possible to add extra demands or circumscribed materials to the performance. Which aspects of performance have to be fulfilled is not completely written down. An example of the aspects of performance is put in ISO 6241(ISO 6241, 1984). In this standard are fourteen aspects defined. These fourteen aspects of an international standard have been put in the Dutch Bouwbesluit (Building Decree). The fourteen performance aspects are divided over five performance fields: safety, utility, health, energy and environment.

### 2.2 Materials

Choice of materials is one of the manners to influence the environmental burden. Extracting raw materials, production efforts, and deterioration of the material are examples of it. Choice of the materials is a determinant for the environmental burden of a building. The way of combining materials, exposure to external conditions and access to components for maintenance have a huge impact on the environment. If demountable joints are being used, the materials can easily be separated. Demountable joints also make it easier to work during maintenance or replacement.

### 2.3 Lifespan

The life span of components is an indication for how long the component can be used. The disadvantage is that lifespan is not a fixed item, but can vary through time. Moreover it is not clear which life span to take. There is a difference between the technical life span, functional life span and economical lifespan. The last two life spans, functional and economical, are often regarded as one. The functional lifespan indicates the period for which a product is desired in a particular situation. The technical lifespan indicates the period that a certain product can fulfill its function. The technical lifespan is much easier to define than the functional lifespan, because the functional life span is much more subjective. This is because the technical lifespan is caused by physical processes, which can be estimated more easily. Life span is one of the bottlenecks within this research.

### 2.4 Use

During the use of a building, the energy consumption and the measures during use (maintenance, replacements) are causing a considerable environmental burden. This is caused by the early replacement of components, but even without these replacements the use of a building causes a burden. In some cases the environmental pressure during the use of a building will be much higher than the initial environmental burden that (will be) caused by constructing and demolishing the building. The alterations in the building are the greatest sources of environmental burden. Installing components that are replaceable in a demountable way can reduce this. The necessary replacements (whether technically necessary or functionally necessary) can easily be conducted without a lot of breaking and cutting. The dismantled components are then ready for reuse or disposal.

Office buildings are much more liable to alterations and for that reason they then have a relatively higher environmental burden during use. The weighing model concentrates on the building of houses, because the market for houses is bigger. However, this does not mean that the weighing model will not be suited for office building as well.

Consequently, the components used and the way these components are placed in the building influence the environmental burden. The applied dimension of the materials is important, in particular the places where connections are made. Construction in that case, goes back to the 196s with construction rules such as patterns and design rules. This does not advocate going back again, but it points out that fixed positioning of joints may have its advantages. With today’s construction techniques (materials, tools, techniques), solutions for a lot of problems can be found.

### 2.5 Optimizing

‘Constructing will always be a necessity’ is one of the statements in the introduction. That different performance factors are required for building a home is clear as well. The performance factor that will be emphasized more than before, is the environmental burden. A couple of years ago, no one cared about the environmental aspects of constructing. But with increasing environmental awareness, this changed. This resulted in the wish (or even demand) to be able to judge a building on its environmental performance as well as on any other kind of performance. With this assessment method, buildings or components of buildings can be compared with each other in such a manner that their environmental burdens and the differences between them become clear.

To make this comparison possible, an assessment of the burden on the environment must be made. Worldwide there are various methods available. A method that is widely used is the Life Cycle Assessment (LCA). The Center of Environmental Sciences of Leiden University (CML) has developed an evaluation method on this subject (Huppes, G, Schneider, F, 1994). To make this method accessible to more people, there are computer programs to support the evaluation. One of these programs is Sima Pro (Pré consultants, 2001). The CML-method is used as a basis for this program. To enhance this method, it is customized for use for architects, constructors and principals in a program called Eco Quantum (Stichting Bouw Research (SBR) and Stuurgroep Experimentele Volkshuisvesting (SEV), 1999). It combines materials into building products. The input is easier to collect this way. The underlying method is the same as with Sima Pro, it even uses the same database. In Eco Quantum the database is much smaller than in Sima Pro and only consists of building materials. This makes it sometimes difficult to put a whole building in the program, because sometimes a product is not available in the database. Sima Pro, on the contrary, has a large database (actually a lot of databases) and has the opportunity to add materials. Every (raw) material is inserted into the database and components can be put together by using these materials. This makes the program suitable for research, but less suitable for the average use by architect or constructor. These methods are suitable for computing fixed data. Every method has its own (dis-)advantages. The output that will be available is important, but the program will provide for that. Once the specifics of the method are known, one can interpret the outcome. However, more important is the input of the program. The input (i.e. scenarios) on which the environmental burden is based is not very fixed when regarding a building.

For optimization, the results of a LC-calculation are taken into account. However, it is not the results of the calculation that are new, but the input used for this calculation. Different aspects form the quality of the building. The extent of conveniences has an influence on the appreciation of the building. The combination of weighing the environmental burden and the desired convenience has to lead to the choice of building(type), detailing of the building and alterations during the entire lifespan.

The scenarios, materials used, and all the assumptions over a long time must be described. There lies the challenge for getting a really accurate Life Cycle Assessment.

## 3 THREE BOTTLENECKS

To achieve a complete weighing, a lot of aspects are necessary. In the previous chapter some of them are mentioned. Before the weighing of the environmental burden becomes available as a 'tool', there are some bottlenecks that have to be widened. These three bottlenecks (conveniences, lifespan and LCA-calculation) will now be illustrated.

### 3.1 A closer look at conveniences

Contemplation of the environmental burden of a building is possible, although there are some obstacles that make it hard to find the right approach. Different techniques, calculation methods and databases are possible to use. To work with these methods the input has to be known. But that is also the crucial question: what does one want?

When the materials are known, then this can easily be calculated. The real question lies before this point and is about products and how to handle the different life spans of products. Are they being used for the full life span, have they been replaced before, or have they even been used longer than the expected life span? An accumulation of these choices doesn't make it easier to predict the environmental burden. This will be dealt with further on.

The influence that the occupant has is enormous. Today it is not the technical lifespan that *indicates* when something has to be replaced, but it is the occupant that *decides* that the (functional) lifespan is over. This behavior is strongly influenced by trends, society and other opinions. This is something that is hard to predict and is linked to behavioral sciences. Their contribution is essential to predicting a probable scenario.

In this paper, the expression 'convenience' is used to indicate the occupants' influence on the building. The question remains what convenience is. It is something everyone understands, but few can give a solid definition. It seems that convenience is when something performs better than the standard. Buildings with 'more' convenience are the isolated buildings for example. When buildings are altered, one is trying to bring in more convenience as well. Convenience can be associated with providing more than standards demand or more than one is used to having at the moment.

Another aspect of conveniences is that they are scaled. It is possible to have 'more' or 'less' convenience. Whether this is only a choice of 'more' and 'less' or if there are different classes is not clear.

The weighing model provides, a solution by describing the performance factors the building has. These factors indicate, among other things, the conveniences. The performance one demands of the building have different aspects within them: safety, usefulness, health, energy and environment. Convenience is a part of these criteria, although it does not become clear right away. In the standard ISO 6241 fourteen performance fields are described (fig.1). Understanding these performance fields, and therefore understanding the roles they fulfill, one can make assumptions about the quality of the building. This quality can then be translated into conveniences.

| **ISO 6241 Performance standard building principles for their preparation and factors to be considered-** | |
|---|---|
| 1. Stability requirements | 8. Visual requirements |
| 2. Fire safety requirements | 9. Tactile requirements |
| 3. Safety in use requirements | 10. Dynamic requirements |
| 4. Tightness requirements | 11. Hygiene requirements |
| 5. Hygrothermal requirements | 12. Requirements for the suitability of spaces for specific use |
| 6. Air purity requirements | 13. Durability requirements |
| 7. Acoustical requirements | 14. Economic requirements |
| | 15. (Environmental requirements) |

**Fig 1: Performance Fileds**

### 3.2 A closer look at life spans

Life span was mentioned before. It is one of the determining issues, perhaps the most determining issues, in making a statement about the environmental burden. Different life spans make the whole aspect even more difficult. But how does one incorporate several sequential life spans of a product? Replacement of building components is something that frequently happens during the use of a building. But what happens if a building can last for 50 years and a component can only last for 15 years. Will the component be replaced after three replacements? Will the last component be used for five more years? It will get even more difficult when the expected life span is not reached. Take a look at a product with a 15-year life span, but with a deviation of 2½ years. In figure 2 this is visualized.

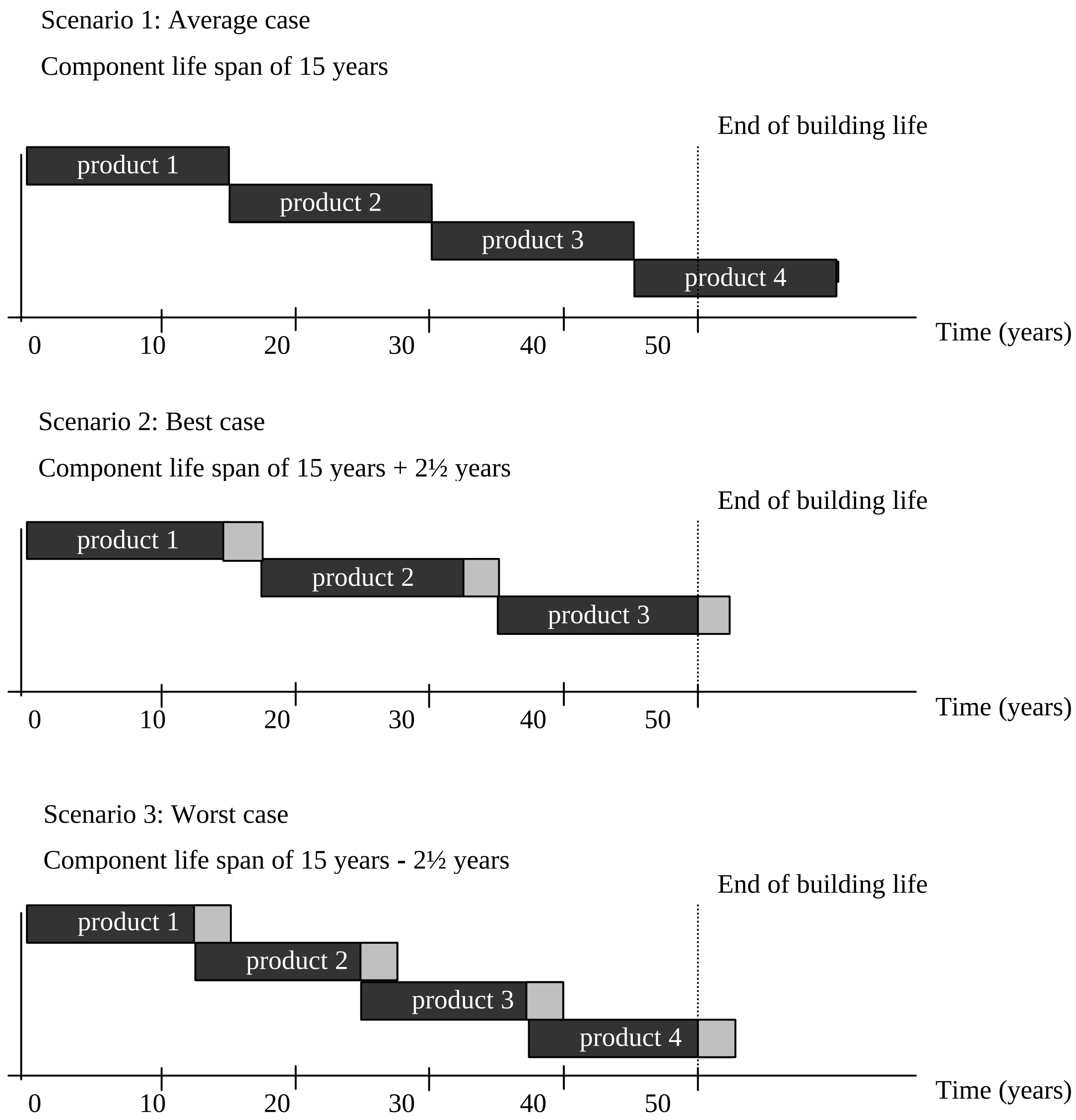


**Fig. 2: Three different life span scenarios**

Three different scenarios use different numbers of components. The average scenario uses four components, but at the end the last product has not been completely ‘used up’. The best scenario only uses three components and can still go on for a longer period. The worst-case scenario fits exactly, but it has to use four components to provide the function. In real-life situations, the various scenarios happen in random order, so to predict the life span of components is a problem.

The reviewed component can consist of smaller parts, of which each part has its own life span with the same difficulties as the component itself. The component is a part of the complete building and in that role defines part of the life span of the building.

Research into this subject is not new. The thesis ‘Deterioration characteristics of building components’ (Hermans, M.H. 1995) indicates how the performance of building components through time can be displayed. With this deterioration characteristic it is possible to connect environmental burden to the building.

To indicate the environmental burden of a component, the initial environmental burden ($M_i$), the end environmental burden ($M_e$) and the environmental burden during use ($M_u$) have to be known. Together they form the total environmental burden ($M_t$).

However, this $M_t$ does not include the time the component is used, and therefore an average environmental burden cannot be circumscribed. The length of the life span refers back to the problems mentioned above. Two components with the same $M_t$ can have a totally different average environmental burden because one has a life span of 10 years and the other has a life span of 20 years (fig. 3).

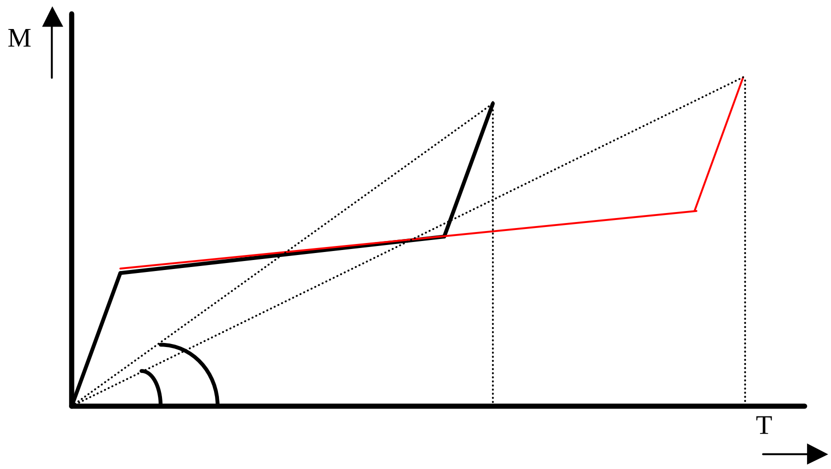


**Fig. 3: Equal environmental burden, different life span**

$M_i$ consists of the environmental burden raw materials have. This includes mining, production, transport, mounting/building. $M_u$ is divided into two separate parts, direct and indirect. Directly related to the component are activities such as painting, maintenance, cleaning. The environmental burden they have is directly caused by the product. The indirect aspects are visible when regarding a building as a product. A well-isolated façade will cause a much lower energy (= $M_u$) demand when compared to a glass wall. This can only be reviewed when the whole house is reviewed as a functional unit.

$M_e$ consists of demounting, transport and processing of waste (waste/reuse scenario's)(fig.4).

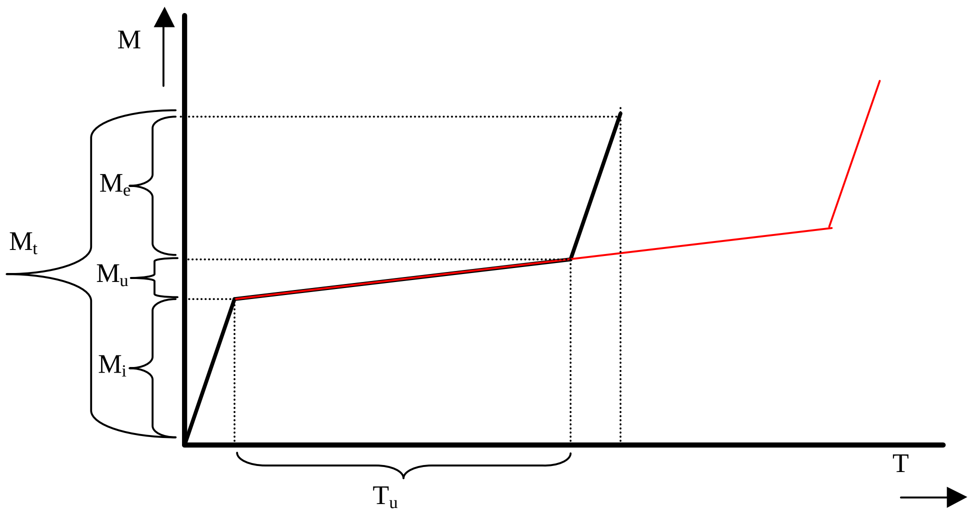


**Fig. 4: Aspects of environmental burden**

The environmental burdens of $M_i$ and $M_e$ are more or less to be known and because of the known impacts the environmental burden can be predicted. $M_u$ , on the contrary, is difficult to control. In the Netherlands, life spans of components are collected by practical experience (Huffmeijer, F.J.M, *et al*., 1998). These values are averages of the building practice and so far are not in any way scientifically defined values. In other countries, using practical values also approaches life span. The life span is of great importance when evaluating the environmental burden. The longer the life span is, the larger $M_u$ will get. However, the longer the life span, the lower the average environmental burden.

So far, only technical life span was regarded. Components consist of materials that are exposed to external conditions, and are degraded by those conditions. These conditions can be modeled and simulated and therefore be predicted. Physical aspects are the main influences in this.

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There is a growing factor that changes the replacement of components: humans. Often components are being replaced before this becomes technically necessary. When this is the case, it is said that the economical or function lifespan has come to an end. To avoid confusion, the expression 'functional life span' will be used in future. The determinant for this type of replacement are the wishes of occupants to change their home. When using the terminology that is introduced in chapter 3.1, it can be said that this is a wish for convenience. In most cases, the interior-related building components are the ones that are replaced, like kitchen and bathrooms. But also the appearance of buildings (facades, extensions, additions) is subject to early replacements. Combined with these activities it can be necessary to adapt other components (i.e. construction) to fulfill the desired change. Contrary to the technical life span, it is not quite possible to predict the period of a functional life span; the behavior of people is too unpredictable for that.

When knowing al these problems still it is desirable to make an assessment of the environmental burden. To make this work, the desired convenience has to be translated to performance fields. Using ISO 6241, convenience can be translated to actual performance fields. When expressing components in so-called 'convenience performances' it is possible to compare different components. 'Convenience performances' are circumscribed by technical performances, which in turn can be materialized. To materials, the life span and the environmental profile of the specific material can be attached. Today's problems are with defining the life span and the definition of convenience, so that the link can be made to used materials and finally the environmental burden.

### 3.3 A closer look at Life Cycle Assessments

Concerning the evaluation of buildings with Life Cycle Assessments (LCA's), there are a few practical problems. Keywords like allocation, classification and data reliability are often discussed. There are various methods that can be used to evaluate buildings and each of them has its own advantages and disadvantages. A real solution for these problems has not been found yet. The software programs have their own basic assumptions and they are included in the program. In the more sophisticated programs, these assumptions can be changed.

Analyzing the processes offers an opportunity to get to know the various influences of processes on the environmental aspects. Making an inventory of materials and processes indicates which emissions occur and what the total environmental burden is. This manner of analyzing is widely used in the industry, and with different purposes. LCA can be used in various situations:

1. non-comparable product studies (process related)
2. comparable product studies (comparing product A and B, both with an equal function)
3. product developing of different products (designing for a function)
4. strategic environmentally based goals (laws and regulations)

LCA's are about comparing products and/or processes to improve them. Especially at the level of a single product this is a fine tool for making decisions. All four of these types apply to LCA purposes within the building sector. (1) Non-comparable product studies concentrate mainly on processes during production and realization. (2) Comparable product studies are of importance when making design decisions such as materialization or building concepts. (3) Product developing studies can be compared with the studies of the building process, but in this particular case it focuses more at components and not at the building as a whole. (4) Strategic environmental based goals will within time be used by government to regulate environmental burden by demanding maximum values. At the moment these studies are used as internal means to set goals.

Within industrial processes this is comprehendable. Products have a life span of only several years, and life spans of five or ten years are mostly exceptions. When regarding the constructing sector, this is of a total different order. Buildings are constructed for 75 years or even longer. They consist of components that have shorter life spans, but they are part of the complex system a building actually is. This long period makes it hard to make statements about the environmental burden a building has during its whole life span. Despite these obstacles, decisions on environmental grounds have to be made. The existing LCA-methods can be used for this, but the input that is used when regarding a building is different to the ones used when regarding industrial processes. By means of the weighing model, the preconditions of an analysis are correctly given.

Not only the input is a bottleneck, the reliability of the data is a problem as well. When manufacturers make their own data it can be questioned whether it is reliable. The way the data are distributed is a problem as well. When data have to be compared, the origin of this data must be the same. Despite the problems, this kind of evaluation has a great future. Compatibility is a requirement that must be achieved in advance. Worldwide standardization of LCA makes it easier to compare components and/or buildings.

## 4 IFD-TODAY

### 4.1 System

The assessment of buildings, not only the performances we review today but on a broader scale (i.e. environment), will ask for a certain transition. Innovative ways of building make it easier to replace components by using simple connections or easily accessible places. In the Netherlands innovative construction methods are stimulated by a government program, called the IFD-program. This stands for Industrial, Flexible an Demountable constructing.

One of these ‘IFD-projects’ is IFD-Today. It is a collaboration between a housing corporation (Amnis, Utrecht), a contractor (Royal IBC), a fitting company (Stork Installatietechniek) and a knowledge center (Eindhoven University of Technology). The aim of this project is to provide a replacement of the existing postwar houses as described before. In this case the existing buildings will be demolished until the foundation is reached. On this foundation a new building will arise, but with a specific building technique. Especially in the soft grounds of the Netherlands, by reusing the foundation a reduction of costs and environmental burden is achieved. Instead of using concrete or brickwork (as used in traditional buildingconstruction), in the IFD-Today project a steel structure is used. This steel frame does not only provide the necessary strength and stability, it also works as a shaft for the piping and wiring of the building. Every part is transported to the building site as a component. That way the building activity turns into a mounting activity, and the construction time can be diminished. This results in shorter temporary housing to be provided by the housing company, less inconvenience for the neighborhood en less interest loss during the construction time. Another advantage is that all the components are mounted and therefore the flexibility of the building is large. When additions are wanted, a component can easily be removed or replaced. These advantages are not only present during the construction time, also during the use of the building it remains an Industrial, Flexible and Demountable building.

The steel construction spans from front to back, without any columns or bearing walls. This makes it possible to make different layouts of the building. In fact, it is also possible to define the size of the house yourself. Within the floors the complete piping and wiring is present. The top layer of this floor can be removed and the piping and wiring can be modified. This is all possible during the use of the building or when a new occupant arrives. It is even possible to move complete building separation walls. This is a measure, which can best be supervised by the housing company, but it is a real possibility. Mainly through its flexibility, the building can be designed at this moment and changed in due time without high costs or environmental burden to the kind of building desired at that moment.

### 4.2 IFD-Today as an example

The IFD-Today principles are realized in a prototype building that has been built in the ‘DuBo-park’ at the Eindhoven University of Technology (fig 5). In this park prototype research can be deducted. The aim of this prototype was to detect if, with the new techniques, it is possible to construct a building that complies with the requirements.

**Fig 5: Prototype at Eindhoven University of Technology**

The building consists of four modules of 7,20m x 11,60m each, and was realized in only six weeks. Because various tests were conducted four different facades and two different floors were used. The possibilities of finishing the building with different claddings are shown this way too. These facades are more or less in accordance with the IFD-principles. They were, however, not the main issue of the project. The floors and the structure are the main issues. The IFD-Today prototype serves as a test site for many researches, varying from organization profiles to contruction matters. In this project it serves as a reference building for an environmental optimized building.

It is to be expected that adaptations in due time can be realized without lot of difficulties. Because of the existing flexibility the wishes of the occupants (convenience) can easily be achieved. This flexibility is possible by using different techniques and materials from those used in traditional building. Instead of a concrete structure a steel one is used. Steel has a higher environmental burden than concrete, but less material is used. These differences are reviewed with LCA-techniques.

The development and realization of the prototype indicates that there is interest in constructing buildings that are easily adapted. Prerequisite known factors are the life span expectancies (both technical and functional); these will be determined using scenarios. These will be made for this project. Then the weighing model can be used and finally the goals of the IFD-today project concerning the environment can be checked and compared to other buildings. Further developments can be pointed out as well.

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## 5 CONCLUSIONS

The way a building is designed, in terms of type and materials, determines the environmental burden of that building. Possibilities to make adaptations during the life span increase the burden so that the final environmental burden can be determined. The presumption is that changing the way of constructing improves the quality of the building through time, especially when the environmental burden of a building is considered. By using other techniques than currently, buildings can be more easily adapted to the wishes of the occupants.

To assess this presumption a weighing model will be developed. The different aspects that are of importance (performance, materials, life span, conveniences, LCA, replacability, optimizing) will be part of this weighing model. The internal relations between different inputs (described as convenience at this moment en through time) and the interpretation of the exploitation periods (life span) are most important. This has to be translated to (existing) techniques to conduct a Life Cycle Assessment. When this link is made, buildings or building components can be compared. This comparison can facilitate designers and decision makers in their design choices.

In the case of this research such a comparison will be made between IFD-Today and a traditional Dutch home. The presumptions made in advance can then be checked and the IFD-project can be optimized.

Weighing between buildings, building types and materials is possible and the option with the lowest environmental burden, that satisfies the conveniences of the occupants, will be built. By doing so the contracting sector will achieve their part of the total reduction of environmental burden.

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# Landscape Evaluation And Improvement Of Sidewalk Concrete Blocks Deteriorated Under The Cold Environment

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**Summary: The concrete products in the sidewalk, like curb block, gutter, cover of gutter, and so on, were often deteriorated by not only the freezing and thawing action but also the salt action of scattering the deicer salt to melt snow and ice on the road under the cold environment. Sidewalk photographs with four ranks from new to deterioration were retouched by the computer graphic technology using the actual condition of road with sidewalk. From the landscapes evaluation questionnaire by four sidewalk photographs for 407 examinees, obviously, the sidewalk with deteriorated curbs indicated the worst rank and the improvement of curb concrete quality needs to keep a driving-safety. In the road with or without scattering the deicer salt, the curb blocks with 39.5, 44.5 and 49.5% of W/C and with or without the treatment of waterproof agent on curb's surface were tested in the actual road. In spite of W/C difference and the waterproof agent, there were many popouts on surfaces of curb blocks in the road scattered the deicer salt in wintertime. The countermeasure to improve the sidewalk landscape must be studied about the durable concrete products.**

**Keywords. Cold Environment, Curb Concrete Block, Freezing And Thawing Action, Landscape Evaluation, Salt Action**

## 1 INTRODUCTION

The concrete products, like curb block, gutter, cover of gutter, and flowerpot in the sidewalk, were often deteriorated in the snowy area of Japan. The phenomenon of concrete deterioration indicates surface cracks and scaling, popouts of mortar and aggregate, many horizontal cracks, and block failures. These concrete deterioration are caused by not only the freezing and thawing action but also the salt action after scattering the deicer salt under the cold environment. The staff of road maintenance said the deterioration of sidewalk concrete blocks has increased by scattering the deicer salt remarkably. Since the regulation to control the utilization of car-tires with studs on the road was announced in April 1991, they have maintained the snowy roads by not only taking out of snow from the road but also scattering the deicer salt. The deicer salt scattering is treated mainly the road where cars were easy to skid on compacted snow and ice on the slanted and/or curved road, the road above bridges, the road without sunny, and on the intersection.

However, if the sidewalk has the deteriorated concrete products, the traffic safety on the road is not kept enough and the road landscape from the drivers' view must not be preferable actually. Hosokawa *et al*. (1996) reported the natural landscape is important in the questionnaire about farm road, and it is estimated the road landscape with deteriorated and poor sidewalk is not preferable. Consequently, it is necessary to study about the landscape evaluation of sidewalk with deteriorated concrete blocks and about the improvement as repairing by using new curb concrete blocks.

## 2 MATERIAL AND METHOD

### 2.1 Landscape evaluation of sidewalk with deteriorated concrete blocks

The various backgrounds in the photograph give many evaluated items to the examinees in the landscape evaluation questionnaire, and it is difficult for the examinees to evaluate the specified parts to be compared easily in the photograph. Therefore, in the case of retouching photographs for the questionnaire, it is necessary to retouch the specified parts only to be compared with the normal photograph while being same background in the photograph. The sidewalk photographs with four ranks from more preferable to very poor were retouched by the computer graphic technology based on the normal road constructed with asphalt concrete and the sidewalk with a little deteriorated curb concrete blocks and some weeds. In the landscapes evaluation questionnaire, four photographs as shown in Fig. 1 were used for 407 examinees as shown in Table 1.

**Table 1. Properties of 407 examinees ( ) : %**

| Sex | | | Age bracket | | | Job | | |
|---|---|---|---|---|---|---|---|---|
| Men | 267 | (65.6) | 10-19 | 39 | (9.6) | Agriculture | 5 | (1.2) |
| Female | 140 | (34.4) | 20-29 | 141 | (34.6) | Business | 10 | (2.5) |
| Total | 407 | (100.0) | 30-39 | 76 | (18.7) | Employee | 154 | (37.8) |
| | | | 40-49 | 84 | (20.6) | Public service | 98 | (24.1) |
| | | | 50-59 | 44 | (10.8) | Member of organization | 24 | (5.9) |
| | | | 60-69 | 17 | (4.2) | Housewife | 23 | (5.7) |
| | | | 70 | 6 | (1.5) | Student | 63 | (15.5) |
| | | | Total | 407 | (100.0) | Other | 30 | (7.4) |
| | | | | | | Total | 407 | (100.0) |

The background in each photograph is same except for the curb concrete blocks in the sidewalk. The normal photograph was taken at the height of 1.16 m as the eye heights of driver and assistant driver above the asphalt concrete pavement. Photograph I indicates the sidewalk constructed with new curb concrete blocks, and Photograph II is the normal sidewalk landscape. However, Photograph III indicates the sidewalk landscape with the broken curb concrete blocks and the aggregate from their blocks, and Photograph IV is the sidewalk with the new curb repaired partly and the old curb.

At the questionnaire, the examinee selected the first watching point in 25 cells devided in each photograph and then chose one from four evaluation items, i.e. “More preferable”, “Preferable”, “Ordinary” and “Poor”, in each photograph. Furthermore, the same questionnaire was occurred to select the first watching point in 25 cells of Photographs V and VI, the view from the driver’s seat (right wheel in Japan) or the assistant driver’s seat (left) by the group of driving usually and the no driving group, respectively. Then, the examinees were required the reason why they chose the four landscape items in each photograph, and the opinion how they thought about the landscape improvement of sidewalk of the road.

### 2.2 Improvement of sidewalk with curb concrete blocks

It is necessary to study about methods to improve the sidewalk landscape. As the curb concrete blocks are especially apt to be deteriorated in the snowy area, the factors influenced to the concrete durability must be analyzed. These factors include the concrete properties, the freezing and thawing action and the deicer salt action mainly. To make clear the factor being deteriorated concrete products, the exposure test was planed in the actual roads, where the deicer salt was scattered or not, while constructing the sidewalk with curb concrete blocks. The curb concrete blocks were manufactured in the concrete products plant in July 1998.

**Photograph I: Normal road and sidewalk with new curb blocks**

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 2.0 | 0.5 | 2.0 | 0.5 | 0.0 |
| B | 1.5 | 4.4 | **30.0** | **18.2** | 0.5 |
| C | 0.0 | 11.3 | **11.5** | 2.7 | 0.0 |
| D | 2.7 | 6.1 | 4.4 | 0.5 | 0.2 |
| E | 0.0 | 0.0 | 0.7 | 0.0 | 0.2 |

**Photograph I**

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**Photograph II: Normal road and sidewalk with some weeds and a little deteriorated curb blocks**

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 0.5 | 1.5 | 1.7 | 0.2 | 0.5 |
| B | 1.5 | 3.2 | **17.0** | **15.5** | 1.7 |
| C | 0.7 | **31.0** | 6.9 | 2.2 | 0.0 |
| D | 6.1 | 5.7 | 2.5 | 0.0 | 0.0 |
| E | 0.0 | 0.0 | 1.5 | 0.0 | 0.2 |

**Photograph II**

**Photograph 111: Sidewalk with deteriorated curb blocks**

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 0.7 | 1.0 | 2.5 | 0.7 | 0.0 |
| B | 0.7 | 2.7 | **13.0** | 8.8 | 0.7 |
| C | 0.7 | **30.0** | 6.1 | 2.0 | 0.2 |
| D | **17.9** | 8.8 | 2.0 | 0.2 | 0.0 |
| E | 0.0 | 0.0 | 0.5 | 0.2 | 0.2 |

**Photograph 111**

**Photograph IV: Sidewalk with repaired curb blocks**

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 1.0 | 1.0 | 2.0 | 0.0 | 0.0 |
| B | 1.2 | 3.4 | **15.5** | **8.6** | 2.0 |
| C | 0.0 | **39.3** | 6.4 | 2.0 | 0.2 |
| D | 5.7 | 7.6 | 2.2 | 0.5 | 0.5 |
| E | 0.0 | 0.0 | 0.7 | 0.0 | 0.2 |

**Photograph IV**

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Photograph V: View from the driver's seat (right wheel) in Photograph III

Photograph VI: View from the assistant driver's seat (right wheel) in Photo. III

**Fig. 3. Photographs used in the questionnaire**

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 0.3 | 0.0 | 0.3 | 0.9 | 0.0 |
| B | 0.6 | 0.3 | **29.5** | **11.3** | 0.3 |
| C | 1.5 | **32.2** | 11.1 | 2.1 | 0.0 |
| D | 6.2 | 2.1 | 1.2 | 0.3 | 0.0 |
| E | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |

Driving a car usually

(341 examinees)

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 1.5 | 0.0 | 0.0 | 0.0 | 0.0 |
| B | 0.0 | 4.5 | **26.9** | **9.0** | 3.0 |
| C | 0.0 | **31.3** | 6.0 | 3.0 | 0.0 |
| D | 6.0 | 1.5 | 4.5 | 0.0 | 1.5 |
| E | 0.0 | 0.0 | 0.0 | 1.5 | 0.0 |

No driving

(66 examinees)

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 0.3 | 0.0 | 1.2 | 0.0 | 0.0 |
| B | 0.9 | 2.7 | **15.3** | 1.5 | 0.3 |
| C | 0.0 | **39.8** | 4.1 | 0.3 | 0.0 |
| D | 10.3 | **19.8** | 0.6 | 0.0 | 0.3 |
| E | 1.8 | 0.6 | 0.3 | 0.0 | 0.0 |

Driving a car usually

(341 examinees)

| | a | b | c | d | e |
|---|---|---|---|---|---|
| A | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| B | 0.0 | 4.5 | **14.9** | 1.5 | 3.0 |
| C | 1.5 | **41.8** | 7.5 | 0.0 | 0.0 |
| D | 7.5 | **11.9** | 1.5 | 0.0 | 0.0 |
| E | 4.5 | 0.0 | 0.0 | 0.0 | 0.0 |

No driving

(66 examinees)

**Fig. 4. Distribution of ratio (%) of the first watching point in 25 cells of Photographs V and VI.**

The materials for curb concrete blocks were ordinary Portland cement with 3.16 specific gravity and aggregate as shown in Table 2, and the mix proportion of concrete is three water cement ratios (W/C) of 39.5, 44.5 and 49.5% as shown in Table 3. The steam with the top temperature of 60-65 $^{o}$C cured the fresh concrete in the mold of curb for about two hours, and the curing maturities until the removing molds were 1300-1360 $^{o}$C-hours.

**Table 2. Physical properties of fine and coarse aggregate for concrete**

| Aggregate | Specific gravity | Water Absorption (%) | Fineness modulus | Unit density (kg/m$^3$) |
|---|---|---|---|---|
| Fine aggregate: Crushed sand | 2.88 | 1.13 | 2.61 | 1750 |
| Coarse aggregate: Crushed stone* | 2.91 | 0.57 | 6.58 | 1750 |

- *Maximum gravel size: 10mm*

**Table 3. Mix proportion of concrete**

| W/C (%) | s/a (%) | Slump (cm) | Air (%) | Unit content (kg/m$^3$) | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | W | C | S | G | Admix* | AE** |
| 39.5 | 43.4 | | | 148.5 | 376 | 854 | 1121 | 1.880 | 0.226 |
| 44.5 | 44.4 | 5.0±1.5 | 5.0±1.0 | 150.0 | 337 | 887 | 1118 | 1.685 | 0.202 |
| 49.5 | 45.4 | | | 151.5 | 306 | 854 | 1112 | 1.530 | 0.184 |

* Air entraining and water reducing agent, ** Air entraining agent

The surfaces on some of the curbs were sprayed the waterproof agent by 0.3 kg/m$^2$. After the construction of sidewalk with curb concrete blocks indicating in Table 3, in December 1998, the test-hammer strength against the top of curb blocks by the normal type of Schmidt hammer and the deterioration of curb surfaces were measured and checked out in January 1999 at first and then in October 1999 and 2000, respectively.

## 3 RESULTS AND DISCUSSION

### 3.1 Landscape evaluation of sidewalk with deteriorated concrete blocks

All examinees watched the questionnaire photographs as the driver, and their first watching points spread in all of photographs, however, these points have a focusing in the driving road and the sidewalk. As shown in Fig. 2, when attention the high-ranking in top three of the ratio of first watching point, the examinees watched the distant view of the road in the less deteriorated sidewalk like Photograph I, but the short-range view and the middle-range view of the road in the deteriorated one like Photographs II, III and IV. Thus, it is clear that the examinees found the sidewalk landscape condition definitely while looking the forward driving-road, and especially watching the deteriorated part in the photograph.

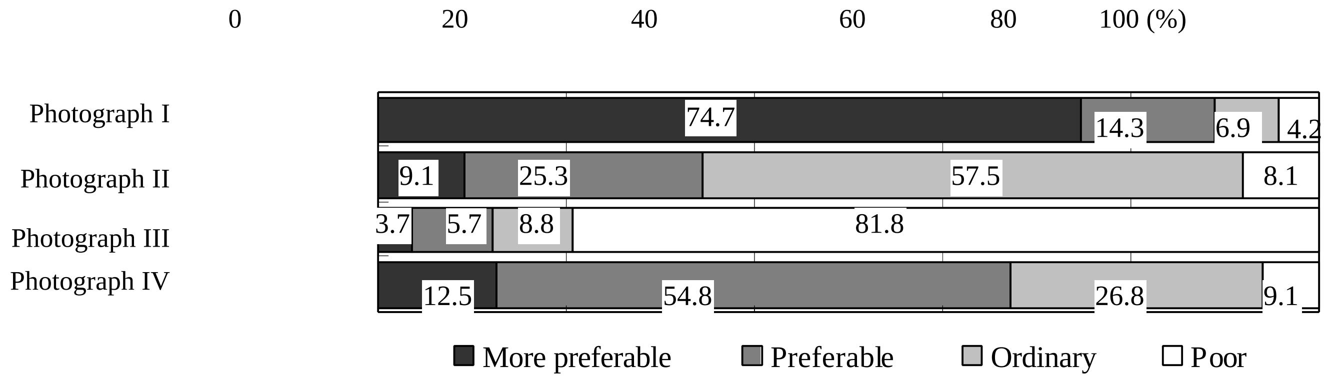


**Fig. 5. Ratio of landscape evaluation in each photograph by all examinees.**

The results of the landscape evaluation of each photograph were shown in Fig. 5. 74.7% of "More preferable" was indicated in Photograph I, 12.5% in Photograph IV, 9.1% in Photograph II, and only 3.7% in Photograph III, and 54.8% of "Preferable" was also in Photograph IV, 25.3% in Photograph II, 14.3% in Photograph I, and only 5.7% in Photograph III. On the other hand, 81.8% of "Poor" was indicated in Photograph III remarkably but 4.2-8.1% in other Photographs. In Photograph II, it was remarkable 57.5% of "Ordinary". In fact, Photograph II shows the normal (ordinary) landscape, and in the result of questionnaire Photograph II was indicated "Ordinary" mainly. It is estimated that the photograph selection in this questionnaire was very suitable. Furthermore, the preferable ranking of landscape from Fig. 5 indicated the top is Photograph I, the second is Photograph IV, the third is Photograph II, and the worst is Photograph III.

Table 4 shows the reasons evaluated each photograph "Poor" while answering number in Photographs I, II, III and IV is 17, 33, 333 and 24, respectively. The examinees to answer in Photograph III were many of 81.8% of all examinees. In the item of the reasons, "Dangerous by deteriorated concrete blocks" were found 4.5% in Photograph III and 47.1% in Photograph II, remarkably. Although "Feel a sense of incompatibility" were found 25-27% in Photographs I and IV, new curbs of sidewalk in Photograph I and also a part of new curbs in the sidewalk in Photograph IV will give incompatible sense to examinees, because of higher the chromatic difference between the new (white color) curbs and the pavement (darker color), as reported by Hosokawa *et al*. (1998), like "Different color of concrete blocks" with 9-12% in Photographs I and IV. The examinees found the weed on the sidewalk dirty in Photograph II. Consequently, the deteriorated sidewalk landscape was evaluated the worst rank and is necessary to improve.

**Table 4. Reasons evaluated each photograph "Poor" by all examinees**

| Item | Ratio* in Photograph (%) | | | |
|---|---|---|---|---|
| | Photo. I | Photo. II | Photo. III | Photo. IV |
| Dangerous by deteriorated concrete blocks | 3.8 | 47.0 | 54.6 | 17.1 |
| Feel a sense of incompatibility | 27.0 | 7.8 | 16.3 | 25.6 |
| In some way | 30.9 | 15.7 | 6.3 | 20.0 |
| Insufficient landscape | 11.5 | 5.9 | 13.0 | 8.6 |
| Dirty with weeds | 11.5 | 17.6 | 3.3 | 2.9 |
| Different color of concrete blocks | 11.5 | 2.0 | 4.9 | 8.6 |
| Non plain landscape | 0.0 | 2.0 | 0.8 | 14.3 |
| Other | 3.8 | 2.0 | 0.8 | 2.9 |

* Answering number in Photographs I, II, III and IV is 17, 33, 333 and 24, respectively.

Table 5 indicates the opinions concerning with the improvement of the sidewalk landscape. In the higher ratio over 20% of opinions, "Road safety", "Improvement of the regional landscape", and "Comfortable driving" are found. Although the choices of items are not many, the result leads to need the quality improvement to keep a road safety. Therefore, for the present, it is necessary to make clear the factors to be deteriorated the curb concrete blocks.

**Table 5. Opinions for the sidewalk landscape improvement**

| Item | Ratio (%) |
|---|---|
| Road safety | 28.8 |
| Improvement of the regional landscape | 28.6 |
| Comfortable driving | 22.3 |
| Attention to the deteriorated concrete blocks | 16.1 |
| Dirty landscape by weeds | 4.0 |
| Other | 0.2 |

### 3.2 Improvement of sidewalk with curb concrete blocks

There are many reports of the durability of concrete durability, but not so many about the concrete products. The factors deteriorate the concrete products include material, vibrating and curing method, constructed place in the water, environment situated products, and so on. Takahashi & Hosokawa (1983) reported the ditches for irrigation and drainage have many kinds of cracks because of wet environment with water and freezing and thawing action. The sidewalk concrete products in the snowy area were often deteriorated by not only freezing and thawing action but also the deicer salt action recently like in this paper. Concerning with the deicer salt action to concrete, some studies were reported by Marchand *et al.* (1994), Rösli & Harnik (1980), Shoya *et al.* (1997) and so on. However, there is no study on the actual road under the cold environment. It is important to make clear the factors to deteriorate concrete by the exposure test on the actual roads, where the deicer salt is scattered or not.

Table 6 shows the results in the exposure test after 2 years. The curbs were usually covered with snow between the middle of January and the last of February. All of the test-hammer strength increased up to 20-40% of the initial strength. In Road A, many popouts on the curb surfaces, as Photographs VII and VIII, in spite of W/C difference and the treatment with waterproof agent. These deteriorated curbs between Roads A and B are obviously influenced by the deicer salt scattering. In Japan, the deicer salt including NaCl and/or $CaCl_2$ mainly is scattered, and actually NaCl deicer salt increases recently by its lower cost. Since we must worry about the influences of ecological environment by the salt, however, the countermeasure against the deterioration of sidewalk concrete blocks must be studied at first.

**Table 6. Surface deterioration and test-hammer strength of curb concrete blocks**

| Road | Deicer salt | W/C (%) | Waterproof agent | Surface deterioration | | Test-hammer strength ratio based on the strength in Jan. 1999 (%) | | |
|---|---|---|---|---|---|---|---|---|
| | | | | Jan. 1999 | Oct. 2000 | Jan. 1999 | Oct. 1999 | Oct. 2000 |
| A | Scattered | 39.5 | Treated | No | Popouts | 100 | 109 | 117 |
| | | 44.5 | | No | Popouts | 100 | 127 | 141 |
| | | 49.5 | | No | Popouts | 100 | 120 | 135 |
| | | 39.5 | No | No | Popouts | 100 | 122 | 128 |
| | | 44.5 | | No | No | 100 | 117 | 120 |
| | | 49.5 | | No | Popouts | 100 | 120 | 124 |
| B | No | 39.5 | Treated | No | No | 100 | 113 | 125 |
| | | 44.5 | | No | No | 100 | 108 | 126 |
| | | 49.5 | | No | No | 100 | 120 | 140 |
| | | 39.5 | No | No | No | 100 | 113 | 120 |
| | | 44.5 | | No | No | 100 | 115 | 123 |
| | | 49.5 | | No | No | 100 | 118 | 132 |

**Photograph VII. Popouts on the surface of curb after 2 years (Road A, W/C: 49.5%, with waterproof agent)**


**Photograph VIII. Coarse aggregate came out after mortar's popout Treatment. in Photograph VII**

### 3.3 Countermeasure to reduce the deterioration of sidewalk concrete blocks

In the snowy area, the deterioration of the sidewalk concrete blocks in the road scattered the deicer salt is called "Salt action damage", severe than the "Frost damage" by the freezing and thawing, and if the reinforced concrete blocks the remarkable deterioration must be occurred. If the concrete blocks were deteriorated within two years after the sidewalk construction, the concrete product company, not the construction company, must have the risk to repair them, because the company manufactured the products be apt to get the damage. The road has been actually maintained to scatter the deicer salt to reduce the traffic accidents in wintertime by the road maintenance office. While being recommended below 45% of W/C in the salty environment by the concrete standard manual of Japanese Society of Civil Engineering, the special countermeasure against the salt action is to spray or paint the waterproof agent on concrete-product surface and to be below 50% of W/C only in the concrete product plant generally.

It is not realistic to manufacture the complete concrete product against both the salt action damage and the frost damage in the concrete plant, and to reduce the deicer salt quantity to keep the lower environmental load in the snowy area of Japan. Therefore, it is suggested that the concrete deterioration is divided the only frost damage and the frost damage with the salt action, from the sidewalk environment. The former indicates the normal concrete blocks can be used in the area without scattering the deicer salt. The latter means that the sidewalk concrete blocks with the special treatment against both damages. However, the place of scattering the deicer salt is decided in the snowy area but the salt is actually brought to the place without scattering the deicer salt by car's splash and strong wind. In general, the concrete products company must treat the special countermeasure on the concrete but without rising cost because there is severe competition among the concrete block manufacturing companies.

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Furthermore, the countermeasure against both the frost damage with the salt action is described in below.

1. We may pay the extra cost for concrete products treated the special waterproof, sometimes the special concrete mix proportion.
2. The development of the deicer agent instead of the deicer salt with $Cl^-$ like a seawater, NaCl, $CaCl_2$ and so on, to keep the lower environmental load.
3. The development of the ecological sand to prevent the car-tire's skid on snow and ice, instead of much of deicer salt scattering.
4. The development of concrete products with zero slump and lower W/C, but with rough surface.
5. It is necessary to make clear the factors of the deterioration of concrete blocks with the deicer salt action.

## 4 CONCLUSIONS

The following conclusions may be drawn from the results of this study:

1. Examinees in the questionnaire found the deteriorated part in the sidewalk definitely. The deteriorated sidewalk landscape was evaluated the worst rank and needs to be improve curb blocks to keep a road safety.
2. The exposure test results on the actual road scattered deicer salt indicated many popouts on curb surfaces in spite of W/C difference and the treatment with waterproof agent after two years. It is estimated that popout phenomenon was influenced by both the freezing and thawing action and the salt action.
3. It is not realistic to manufacture the complete concrete product against both the salt action damage and the frost damage in the concrete plant, and to reduce the deicer salt quantity to keep the lower environmental load in the snowy area of Japan. As the countermeasure against both damage, for example, we may pay the extra cost for concrete products treated the special waterproof, sometimes the special concrete mix proportion, or the development of the deicer agent and the scattering sand to prevent the car-tire's skid instead of the deicer salt.


## 5 ACKNOWLEDGMENTS

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# Carbonation Testing Of Hardened Concrete And The Effect Of Cement Type

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**Summary: Carbonation-induced corrosion affects all reinforced concrete building structures where the moisture saturation level of the capillaries is suitable for supporting both $CO_2$ ingress and steel oxidation, either continuously or cyclically with different seasonal environmental exposure conditions. Such conditions are found in the externally exposed elements of structures and buildings exposed to or sheltered from rain, which accounts for approximately two-thirds of all structural concrete used. The service life of most reinforced concrete building structures is, therefore, normally governed by cover carbonation and subsequent reinforcement corrosion. Although carbonation-induced corrosion is rarely catastrophic, the effects are on serviceability and aesthetics which are economically significant for large-scale property owners, such as municipal housing authorities.**

**With no standardised laboratory method, the European Committee for Standardisation (CEN) established a joint Working Group (TC51/WG12) to develop a carbonation test for hardened concrete. A draft method was developed using the RILEM procedure for measuring the depth of carbonation and was subjected to a inter-laboratory round-robin series of tests to measure its repeatability. The indicated precision from these tests was, however, poor and consequently there was no subsequent agreement to publish the method as a draft for development (ENV) ahead of a full European Standard (EN). However, it was felt that the draft method did provide a first step towards the development of an agreed standard test procedure and was published as a CEN Report.**

**As a result of this initial study, the CEN Working Group identified that further test development was needed. This paper reports the findings of a study of the draft CEN carbonation test method that was aimed at identifying the causes of the poor precision experienced in the round-robin test programme. Using these findings, the study developed improvements to the test method, which are evaluated using as limited series of repeated test mixes. The paper then describes the effect of different cement types on the carbonation performance of concrete. A tentative classification of concrete based on cement type will also be presented.**

**Keywords: Carbonation, Cement type, Environmental Control, Performance.**

## 1 INTRODUCTION

Concrete durability, unlike its structural design properties, is difficult to estimate by indirect means, such as compressive strength or permeation properties. It may be argued that the inability to determine the durability potential of concrete has contributed to the massive current expenditure on repair and maintenance of structures, which corresponds to around £20 bn from an overall construction turnover of £45 bn in the UK alone (DETR, 1999). Such problems are being addressed by the European Committee for Standardisation (CEN), who are developing a series of test methods for determining the durability of concrete exposed to standardised storage conditions, for example in carbonating environments.

Carbonation-induced corrosion affects all structures where the moisture saturation level of the capillaries is suitable for supporting both $CO_2$ ingress and steel oxidation. These conditions are typically found in the externally exposed elements of structures and buildings which may be exposed or sheltered from rain, which accounts for around 2/3 of all structural concrete used (Parrott, 1996). The service life of most reinforced concrete building structures is, therefore, normally governed by cover carbonation and subsequent reinforcement corrosion (Hobbs *et al*, 1998). Although carbonation-induced corrosion is rarely catastrophic (Concrete Society, 1996), the effects are economically significant and repair costs often outweigh new build expenditure.

A draft method was developed using the RILEM procedure (RILEM, 1988) for measuring the depth of carbonation and was subjected to a inter-laboratory round-robin series of tests to measure its repeatability (CEN, 1997). The precision of these tests was poor, however, it was felt that the draft method did provide a first step towards the development of an agreed standard test procedure and was subsequently published as a CEN Report (CEN, 1997).

To enable sustainable and economic reinforced concrete building construction, the reliable determination of the carbonation rate of hardened concrete is essential to allow design decisions to be made. In addition to this, the new European Standards for cement and concrete, namely EN 197-1 and EN 206-1 (including the relevant national application documents), allow the engineer scope to select cement and concrete characteristics and requirements appropriate to local conditions.

In support of this goal, the CTU has carried out a 3 year study with two main aims. Firstly, to identify the areas within the CEN test leading to poor precision and modify the test to become a repeatable methodology. Secondly, study also compared the carbonation of concrete containing 10 different combinations of common cements to observe carbonation rates of concrete subject to different storage conditions, in terms of the degree of wetting applied to the test specimens. The test programme also attempted to tentatively classify the carbonation performance of different concretes.

## 2 DEVELOPMENT OF SIMULATED NATURAL CARBONATION TEST METHOD

The initial work focussed on the development of a simulated natural carbonation test method. Table 1 summarises the key requirements of the current draft CEN test. Storage conditions were divided into 3 exposure classes, so that test specimens had varying pore saturation levels as could be expected to occur in different local micro-climates (BRE, 1995). The test method adopted the recommendations of RILEM CPC-18 (RILEM, 1988) to measure the depth of carbonation, i.e. direct measurement of the neutralised depth of cover using a sprayed on phenolphthalein indicator solution. A pan-European inter-laboratory test carried out by CEN showed that the test had a poor repeatability, however, there was potential for the method to be further developed.

A comprehensive analysis of the inter-laboratory data is presented elsewhere (Jones *et al*, 2000) however the main conclusions arising from the analysis test were:

(i) variations in environmental control and concrete production techniques were heavily influencing the carbonation readings. Only one laboratory conformed to the environmental requirements of a temperature of 20 ± 2EC, relative humidity of 65 ± 5% and an atmospheric $CO_2$ concentration of 350 ± 50 ppm.

(ii) concrete production techniques led to inter-laboratory variations. Although laboratories used local aggregates, differences in intern-laboratory concrete strengths were found as a result of variations in water/cement ratio.

**Table 1. Requirements of the simulated natural carbonation test method**

| | | |
|---|---|---|
| *Specimen Size* | 2 Concrete Prisms: 100x100mm and ∃400 mm long. | |
| *Initial Curing* | 24 hours under damp hessian and impermeable plastic sheeting followed by 3 days seal cured. | |
| *Storage Conditions* | | |
| *Exposure Class 1:* | Temperature: | 20±2EC . |
| | Relative Humidity: | 65 ± 5%. |
| *Exposure Class 2:* | $CO_2$ Concentration: | 350 ± 50 ppm. |
| | As Class 1 but fully immerse test prisms in water every 28 days for 6 hours. | |
| *Exposure Class 3:* | As Class 2 but immerse test prisms in water every 7 days. | |
| *Test Ages* | 28, 91, 182, 273, 364 and 728 days ±2% after start of test. | |
| *Method of Measuring Depth of Carbonation* | 50mm slice broken from prism and phenolphthalein sprayed on freshly broken surface. Measurement made by taking the average of 5 readings made on each face. | |
| *Reporting of Depths of Carbonation* | Average carbonation depths from the two specimens to be reported for each exposure class. | |

To this end, the present study aimed to reduce the variability associate with the test method by addressing the environmental requirements and concrete production techniques

### 2.1 Audit of Test Storage Conditions and Development of Active Environmental Control System

Four different storage environments for the test specimens were investigated, as follows:

S1: Uncontrolled, open laboratory atmosphere with an indirect link to an external air source.

S2: Temperature and relative humidity controlled storage room with an air-tight door closed.

S3: As S2 but with the door propped slightly ajar continuously.

S4: As S2 but with the door fully opened for 24 hours in every 48 hours.

In each case recordings of the $CO_2$ concentration, RH and temperature were made over a period of 1 month, as shown in Fig 1. It was found that the open laboratory storage (S1), did not conform to the $CO_2$ requirements and a controlled climate room with dimensions 2100x1900x2400 mm was used as a storage room to store the test specimens. This room had relative humidity and temperature control (S2) and was filled with ‘dummy' concrete specimens such that the total exposed concrete surface area in the room was 125 m². Although relative humidity and temperature was maintained, the $CO_2$ concentration was depleted considerably, due to the consumption of atmospheric $CO_2$ gas by the concrete. As a result the room door (2 m²) was wedged slightly ajar storage, (S3) to allow exchange of air with the open laboratory however this gave results similar to S1. In turn, storage environment S4 was tried however, storage conditions did not conform to the test requirements. It was evident that it would be necessary to control $CO_2$, RH and temperature concurrently to conform to the requirements of the draft CEN test.

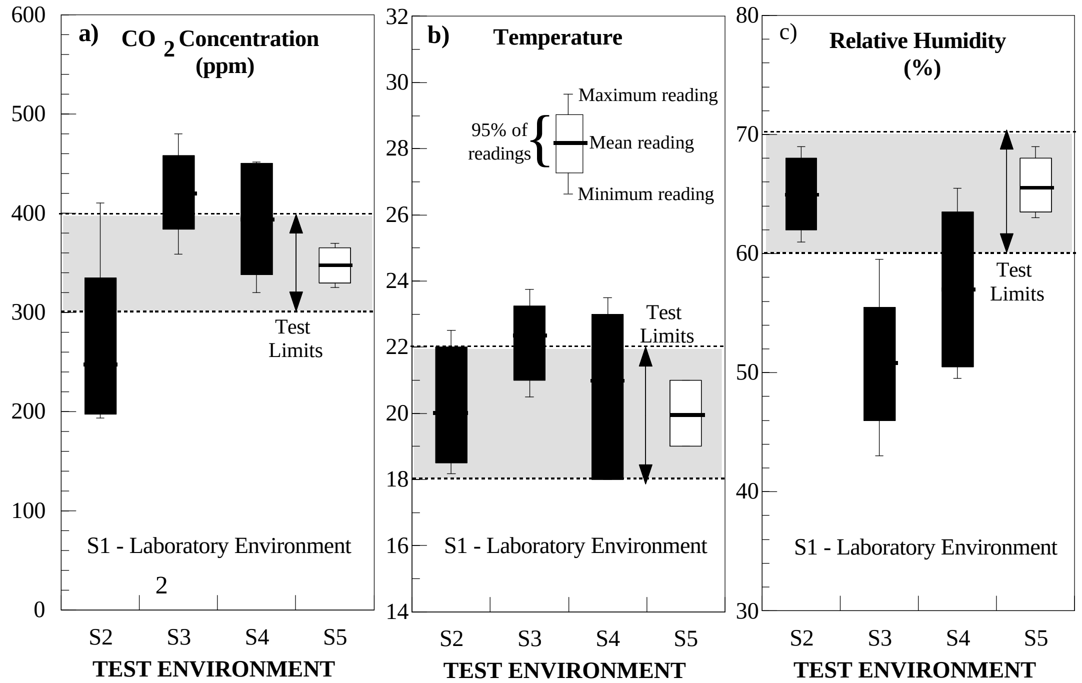


**Figure 1. Box whisker plots of variations in laboratory storage environment**

Proprietary controller and monitoring systems were prohibitively expensive, thus an in-house CO2 controller and injection system was retro fitted to the climate room, Fig 2. The performance of the active control system in the storage room was monitored over a period of 1 month and the environmental parameters were kept within the test limits, as shown in Test Environment S5, Fig 1. Standard table fans were placed within the storage room to provide a turbulent atmosphere and prevent the formation of locally depressed partial pressures of $CO_2$ which can be seen in Fig.2.

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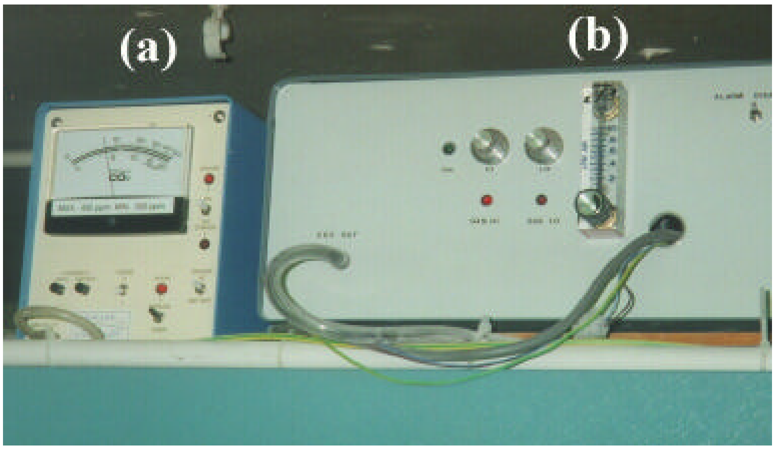


**Figure 2. Retrofitted $CO_2$ system (Top): (a) monitoring/controllor system, (b) injection system. The interior of the modified test carbonation chamber (Right)**

### 2.2 Normalisation Procedure for Determination of Carbonation Depth

Given that the depths of carbonation in the simulated natural carbonation test, after 1 year, were likely to be low and that within all laboratories, a degree of variability in the production of test specimens is inevitable, it was considered necessary to devise and apply a procedure for normalising the carbonation test data. This was felt particularly important if such data is to be used for exact comparison either with other data or against a particular benchmark. Furthermore, if the test data is to be used to estimate long term carbonation depths using mathematical models, initial variability must be minimised. An example of the procedure is shown in Figure 3.

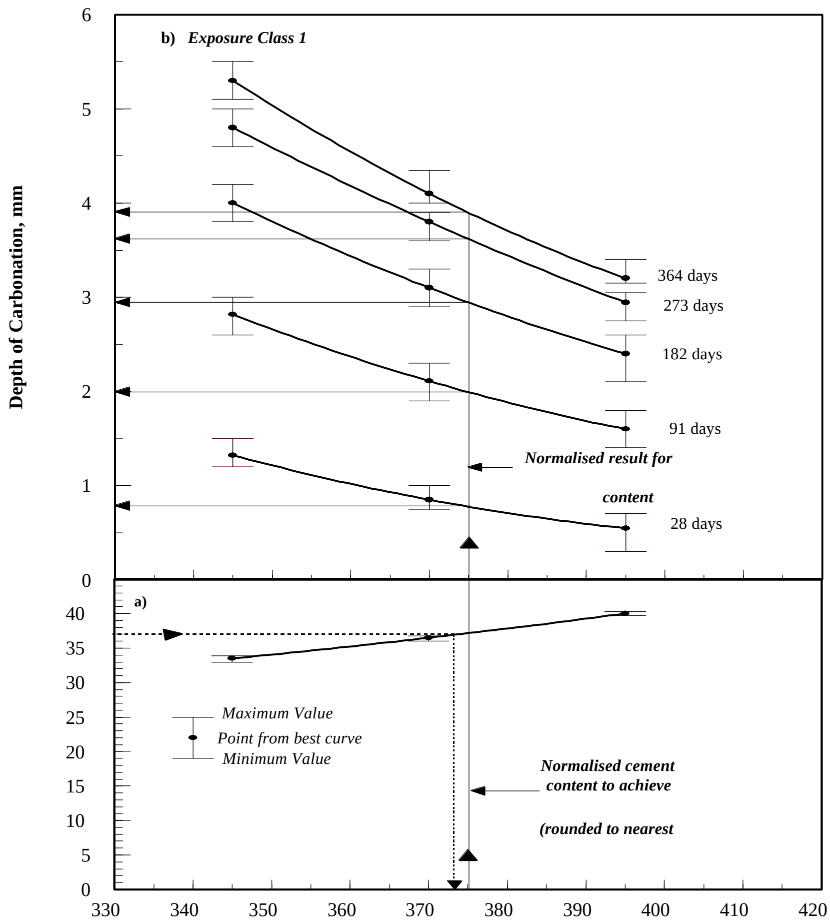


**Figure 3 Normalisation procedure a) 28 day compressive strength is plotted against cement content to find actual proportions for required compressive strength. b) carbonation depth is then determined.**

The normalisation procedure developed used the following mixes:

i) A *primary mix* was designed to achieve the required concrete grade.

ii) *Two secondary mixes* were also produced with ± 8% by mass of the total cement content of the primary mix. This value was arbitrarily chosen but corresponded to ± 25kg/m³ for this testing programme where a 37 N/mm² PC/30% PFA mix was used. The secondary mixes had the same free water content as the primary mix and proportions were volumetrically adjusted by altering the fine aggregate content.

Preliminary work showed that variations in compressive strength between 35-39N/mm$^2$ gave a variation in carbonation depth of 2mm at 1 year. Although this may seem a relatively small difference, when these carbonation depths are projected to 30 or 40 years the error is substantially multiplied.

### 2.3 Repeatability of Modified Carbonation Test

In order to determine the effects of the proposed modifications on the test method, a limited series of 10 repeated mixes giving 2 sets of test samples were produced. One group was subjected to the original draft CEN method testing and the second with the proposed method of testing, with both series of test samples stored in the Dundee storage room to CEN Class 1 exposure conditions. In addition, for comparative purposes two further subsets of test samples were stored for 20 weeks in an accelerated carbonation chamber with an atmospheric environment of 4000 ± 500 $CO_2$ ppm, 50 ±5 % RH and 20 ±2 C (Dhir *et al*, 1989). The results were statistically analysed and at a 95% confidence level there was no significant difference in the range of individual test results. The repeatability of the results was significantly improved when the Dundee test procedure was adopted.

## 3 CARBONATION RESISTANCE OF COMMON CEMENT CONCRETES

The objective of the second phase of study was to compare carbonation rates of concretes containing different cement types and quantities subject to different storage conditions, in terms of the degree of wetting applied to the test specimens. European Standards for cement and concrete, namely EN 197-1 and EN 206-1 allow the engineer scope to select cement and concrete characteristics and requirements appropriate to local conditions. However, the long-term performance of these materials is relatively unknown, especially in terms of carbonation resistance.

### 3.1 Experimental Programme

A total of 10 different cements were selected as summarised in Table 2. The materials were chosen to reflect the array of cement and concrete types which are now in use around Europe, containing materials such as pulverized-fuel ash (PFA), ground granulated blastfurnace slag (GBS), silica fume (CSF) and metakaolin (MK). The mixes were designed using a standard mix constituent proportioning method (Teychenné *et al*, 1997), to achieve a nominal 28 day cube strength of 37 N/mm². Most urban structures now require a concrete grade of between 30-40 N/mm² (CEN, 1997), thus in the experimental programme the primary mix was designed to achieve a characteristic strength of 37 N/mm² after 28 days of standard water curing, corresponding to a cylinder strength of 30 N/mm².

The mixes were compared to a Reference concrete, a PC/30% PFA mix. From a comprehensive review of published and unpublished literature, it was felt that this mix was now common place and would provide a substantial degree of carbonation resistance against which performance may be judged. However, the Reference concrete may be any concrete that is deemed suitable by the user to allow real judgement of the performance of various test mixes.

**Table 2. Mix proportions for test mixes, normalised to a standard cube strength of 37N/mm²**

| *Cement Type* | *Normalised Mix Constituent Proportions(1), kg/m³* | | | | | *Plasticiser Dosage, l/100kg Cement* | *w/c ratio* |
|---|---|---|---|---|---|---|---|
| | *Total Cement* | *Water (2)* | *Aggregates* | | | | |
| | | | *Sand (3)* | *10 mm (4)* | *20 mm (4)* | | |
| **Reference Mix** | | | | | | | |
| PFA 30% | 375 | 170 | 640 | 400 | 800 | none required | 0.46 |
| **Test Mixes** | | | | | | | |
| *Portland Cement* | | | | | | | |
| PC(5) | 300 | 185 | 700 | 400 | 800 | none required | 0.59 |
| *Blastfurnace Slag* | | | | | | | |
| GBS 40% | 325 | 180 | 680 | 400 | 800 | none required | 0.52 |
| GBS 50% | 365 | 180 | 640 | 400 | 800 | | 0.48 |
| GBS 65% | 390 | 180 | 625 | 400 | 800 | | 0.47 |
| *Metakaolin* | | | | | | | |
| MK 10% | 315 | 185 | 685 | 400 | 800 | 0.55 | 0.61 |
| MK 15% | 285 | 185 | 715 | 400 | 800 | 0.62 | 0.63 |
| MK 20% | 280 | 185 | 720 | 400 | 800 | 0.63 | 0.64 |
| *Silica Fume* | | | | | | | |
| CSF 10% | 280 | 185 | 720 | 400 | 800 | 0.60 | 0.65 |
| CSF 15% | 285 | 185 | 715 | 400 | 800 | 0.61 | 0.61 |

(1) The desired proportions to achieve an exact 28 day strength of 37N/mm$^2$.

(2) Free water content to achieve a slump of 60-90mm. Plasticizer used as shown.

(3) Natural sand, zone M grading to BS 812:1992

(4) Coarse aggregate: natural gravel

(5) Portland Cement is strength class 42.5 N to BS 12 (1996)

### 3.2 Effect of Cement Type on Carbonation Depth

The modified Dundee test programme was run for 2 years, as it had already been noted that carbonation depths would be minimal at 12 months. For each test prism, 20 individual measurements of the depths of carbonation were taken, giving a total of 40 readings for each cement type in each Exposure Class. The 2 year depths of carbonation for each of the concretes is shown in Figure 4.

Within Exposure Class 1, where no wetting was carried out, the effect of cement type was most noticeable, with PC and MK10% mixes having the lowest depths of carbonation after 2 years. It should be recognised that under these conditions, corrosion is improbable as the concrete is unlikely to have sufficient pore saturation, however, the effect of replacing the PC with PFA, GBS and the higher levels of MK and CSF was found to give increased depths of carbonation.

The Figure also compares the carbonation depths for the different cement types in the 3 Exposure Classes. Storage in Exposure Class 3 (wettest) had the lowest depth of carbonation and highest pore saturation level. Exposure Class 2 resulted in depths of carbonation between Class 1 and 3 due to the period of wetting.

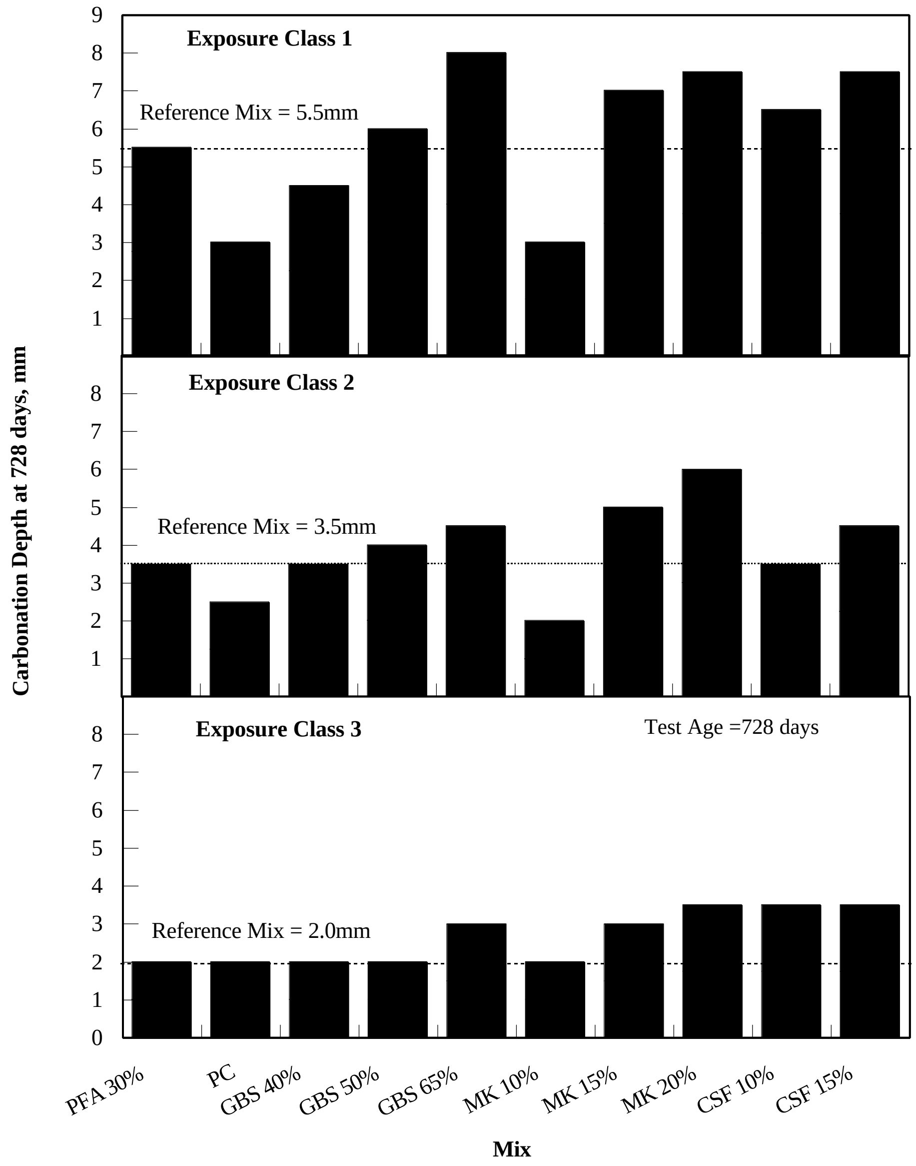


**Figure 4. Comparison of 2 year carbonation depths of common cement types using the Dundee modified test.**

The problem, however, for the engineer is which of these Exposure Classes represents natural exposure environments. At present, there does not appear to be sufficient data to make an informed judgement and there are likely to be a range of exposure conditions that can support carbonation-induced corrosion activity in different geographical locations for example the north-west coast of the UK is considerably wetter than the south-east coast. Although Class 1 is the worst case condition for carbonation, it is considered that this condition is too dry to support corrosion. On the other hand, Class 3 is considered not only wetter than any natural environment but the resulting pore saturation would produce only minimal corrosion activity. It is recognised therefore that further work is necessary to define exposure class environments which simulate the natural exposure environments capable of supporting both carbonation and carbonation-induced corrosion.

## 4 NEAR SURFACE CONCRETE QUALITY AND CARBONATION RESISTANCE

The ranking order of the cement types were similar regardless of the Exposure Class to which they were exposed. Exposure Class 1 results, which gave the worst case with regards to carbonation, had the broadest spread of the cement types and, using an arbitrary division of approximately 1.5mm carbonation to differentiate the cement types, the following tentative grouping can be made:

PC = MK10

GBS40/50 = PFA

CSF10/15 = MK15/20 = GBS65

In Exposure Class 2, the addition of a monthly wetting and drying cycle compresses the spread of results and two groups emerge, which are broadly similar to Class 1:

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PC = MK10 = GBS40/50 = PFA = CSF10

CSF 15 = MK15/20 = GBS65

In Class 3, the weekly wetting and drying cycle results in similar carbonation depths for all the mixes, however, an arbitrary grouping would appear to be:

PC = MK10 = GBS40/50/65 = PFA

CSF10/25 = MK15/20

The carbonation durability ranking of the cement types is likely to be affected by a combination of the permeation properties of the concrete and the degree of cover zone pore fluid alkalinity. Although there is no permeation test that wholly determines the microstructure, accessibility and interconnectivity of the pore system to $CO_2$, a water vapour diffusivity test was used (Dhir *et al*, 1989) to characterise the concrete cover zone. This test was found to be the most sensitive method to changes in microstructural properties and cement type. In addition, it is difficult to determine the cover zone pore fluid alkalinity directly, therefore, the total amount of $Ca(OH)_2$ was measured in the cover zone concrete at the pre-entry (ie prior to exposure to the relevant Exposure Class) and after 2 years.

Table 3 compares the near surface water vapour diffusivity and $Ca(OH)_2$ in the outer 10mm of the concrete. The cement types are ranked in order from the lowest depth of carbonation to the highest in Exposure Class 2, however, it was clear that the GBS mixes were acting differently from the pozzolanic cement combinations.

Although the PC mix had a very high initial water vapour diffusivity, the high $Ca(OH)_2$ concentration of the cover prevented rapid progress of the carbonation front. The $Ca(OH)_2$ content of the cover also depleted slightly over the test period however the concentration remained relatively high at 2 years. The MK10% mix showed the combined effect of a low water vapour diffusivity and high initial $Ca(OH)_2$ concentration. One would expect this to out-perform the PC mix however with time, the $Ca(OH)_2$ concentration depleted, due to a combination of carbonation and the pozzolanic reaction, leading to a similar performance to that of the PC mix.

The remaining pozzolanic mixes, PFA, MK 15%, MK 20%, CSF 10%, CSF 15% showed a relatively similar performance in carbonation. The MK and CSF mixes had a low water vapour diffusivity however the $Ca(OH)_2$ concentration was also very low and depleted with time. Subsequently, the carbonation was relatively similar within these concretes.

**Table 3. Near surface water vapour diffusivity and total $Ca(OH)_2$ content**

| *Cement Type* [(1)] | *Water Vapour Diffusivity, g/mm² s mm Hg x $10^{-10}$* | | *Total $Ca(OH)_2$, % wt of cover zone concrete* | | | |
|---|---|---|---|---|---|---|
| | *Pre-entry* | *2 year Exposure Class 1* | *Pre-entry* | *After 2 year Exposure* | | |
| | | | | *Class 1* | *Class 2* | *Class 3* |
| *MK 10%* | 51 | 45 | 4.3 | 1.2 | 1.9 | 2.5 |
| *PC* | 192 | 154 | 5.2 | 4.3 | 5.0 | 5.6 |
| *PFA 30%* | 112 | 92 | 1.7 | 0.7 | 1.2 | 1.6 |
| *GBS 40%* | 96 | 114 | 2.0 | 0.7 | 1.4 | 1.7 |
| *CSF 10%* | 58 | 34 | 1.5 | 0.5 | 1.4 | 1.7 |
| *GBS 50%* | 128 | 159 | 1.7 | 0.6 | 1.4 | 2.0 |
| *CSF 15%* | 64 | 36 | 1.4 | 0.5 | 0.5 | 1.2 |
| *GBS 65%* | 147 | 192 | 0.7 | 0.4 | 1.2 | 1.6 |
| *MK 15%* | 70 | 50 | 1.6 | 0.6 | 0.9 | 1.1 |
| *MK 20%* | 77 | 47 | 1.3 | 0.6 | 0.5 | 1.0 |

[*] Measured prior to exposure to carbonating atmosphere

There are obvious difficulties with the assessment of water vapour diffusivity and the determination of the hydroxyl ion concentration in the concrete pore fluids. It is hypothesised that it may be possible to estimate the likely carbonation resistance for a particular concrete mix based on the water/PC ratio. Figure 5 plots the water/PC ratio against 2 year carbonation depth and it can be seen that mixes with a higher water/PC ratio generally have higher carbonation depths.

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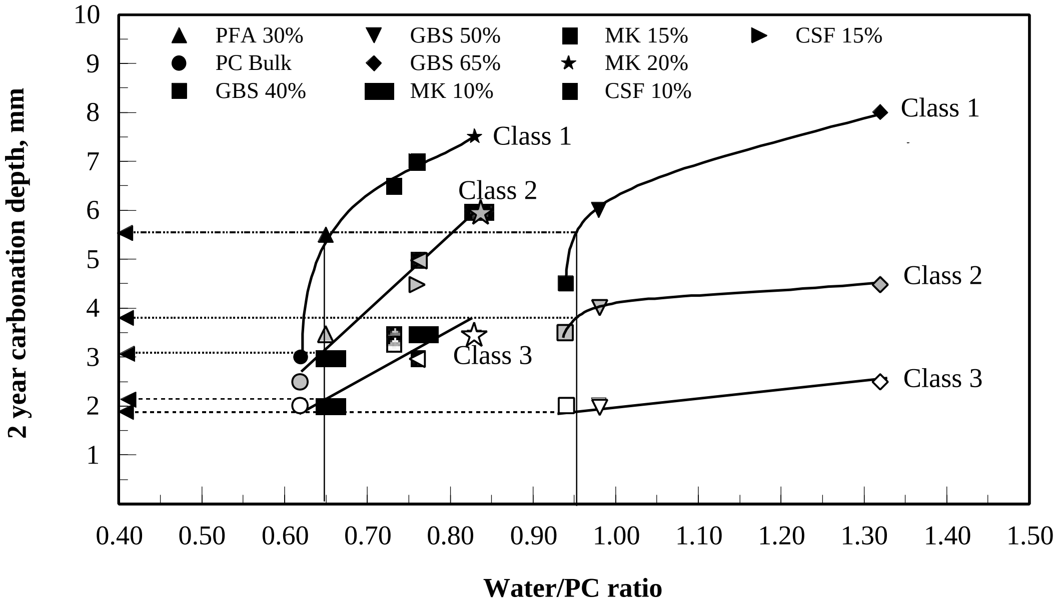


**Figure 5. Relationship between 2 year carbonation and water/PC ratio for Exposure Classes 1, 2 and 3.**

Using the PC/PFA Reference Mix as a benchmark, it is possible to provide a limiting value for water/PC ratio which would result in similar or lower depths of carbonation occurring for any other combination of cements for concrete of the same compressive strength. From the results it is suggested that, at a given level of compressive strength, (in this case 37N/mm²) a tentative limiting water/PC ratio of 0.65 should be specified for mixes containing pozzolanic cements and 0.95 for slag concrete.

## 5 CONCLUSIONS

The current study aimed to develop a simulated natural carbonation test method and determine the effects of various cement types on carbonation resistance. The existing test method developed by CEN was found to have poor repeatability. Poor control over the environmental conditions, in particular the levels of $CO_2$ in the atmosphere was significantly affecting carbonation. Conformance with the test storage requirements could only be achieved through the use of a fully air conditioned sealed room, however, it was necessary to provide actively controlled $CO_2$ injection in order to maintain the required $CO_2$ concentration in the room. Significant depletion of atmospheric $CO_2$ was noted within about 5 days if this was not replenished. As a result, a low cost $CO_2$ monitor/controller system was designed for retro fitting to suitable storage rooms found in most well equipped laboratories.

The production method of concrete test mixes was also affecting the test repeatability and a normalisation procedure for the determination of carbonation depth was developed. By plotting strength against cement content for a single primary and two secondary mixes, the normalised mix proportions to give the test concrete standard cube strength can be determined. Using this value and the binder content against depth of carbonation curves, the normalised depth of carbonation can be determined at any particular test age. A limited series of repeatability tests were carried out comparing the effects of this two stage normalisation process and it was found that the coefficient of variation was substantially reduced.

The modified test method was used to compare cement types in carbonating environments. The results showed the broadest spread of results in Exposure Class 1. Across 10 cement combinations a spread of 5mm was found after 2 years testing. As the frequency of cyclic wetting and drying increased, so the spread of carbonation depth reduced across cement type which was expected. Although Exposure Class 1 may be deemed a worst case scenario for carbonation it is unlikely that corrosion would occur. However, in Class 3 the concrete may be deemed to be too wet to allow carbonation thus it is also unlikely to lead to a corrosion problem. Further work is needed to make an informed judgement as to which Exposure Class is representative of the real external environment, however, results suggest that the level of exposure to moisture may be somewhere between Class 1 (no wetting) and Class 2 (wetting every 28 days).

A further parameter, water/PC ratio showed that, with the exception of GBS mixes, all cement types could be grouped within a specific concrete grade to give a tentative limiting water/PC ratio. For all cement types (with the exception of GBS), a maximum water/PC of 0.65 would give a similar carbonation performance to that of the Reference mix and this was similar in all Exposure Classes. To provide a carbonation performance similar to that of the Reference Mix, GBS mixes allowed a higher water/PC limit of 0.95. Although this limit showed a slight variation in Classes 2 and 3 the limit still gave carbonation depths similar to the other cement types. The potential carbonation resistance of concrete was determined to be dependant on both permeation and alkalinity of the cover concrete. A comparison of water vapour diffusivity and total $Ca(OH)_2$ in the cover concrete showed that concretes with lower water vapour diffusivity properties still exhibited higher carbonation depths due to the fact that the cover concrete alkalinity was relatively low.

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# The Methodology For The Evaluation Of Building Components' Durability

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**Summary**: The use of building components and their exposition to the weathering agents causes degradations during time of their functionality, called "aging", towards which it's necessary to do maintenance works, with consumption of materials and economic resources. It's important to gain know-how on building components' durability and on performance degradation phenomena in order to design buildings characterized by an economically reasonable service life.

**For each built space the indoor conditions will be controlled if the technological performances of the building components that delimit them are maintained, because of the preservation of constituting products' properties. In coherence with this approach the DISET Durability Research Team proposes a method for the evaluation of durability organized into environmental and technological performances aspects, technical and design aspects.**

**As for the two first aspects the method proposes a procedure to predict and quantify on a sample space the indoor performance variations as a function of the components' performance linked to materials characteristics' variations. Then the indoor performance limits are linked to building components performance characteristics.**

**As for the technical aspects, tests are carried out in cooperation with the laboratory of SUPSI (CH) to evaluate effects on building components and materials caused by weathering agents. In laboratory building components samples are subject to accelerated aging and the performances' decay is monitored. When the measured values reach the acceptable performances' limits the final values of characteristics and linked properties of materials are measured. To evaluate the service life outdoor natural aging tests are being developed on samples of the same components, exposed to the outdoor environment and monitored as for the technological performances. The service life is given by the time going from the beginning of sample exposition and the measuring of performance limit reaching.**

**As for design aspects the method has worked out judgement criteria for reliability propensity of designed building components, based on their functional model, the executive complexity, the inherent variability and the chemical –physical compatibility. The research results give support information to building components design for a planned service life. The design application of reliability estimation criteria allows to reduce the risks not to reach the planned service life.**

**Keywords. Durability, reliability, building components, materials, performances**

## 1 INTRODUCTION - THE DURABILITY OF BUILDING COMPONENTS

The methodology for the durability estimation of building components, here reported, is the result of research activity that for years has been developed by the durability working group of Department of Engineering of Building Systems and Urban Planning, Politecnico of Milan, and that derives its methodological set-up from disciplinary considerations on building technological quality, of which the durability is a particular aspect.

The durability estimation consists in the identification, in a technical element, of the attitude to supply the initial performances from its installation, for a fixed time and with planned intensity. The time in which a technical element holds the expected functioning is defined as service life or average time of good operation.

The quality is defined by the national and over-national standard as “the ability to satisfy throughout performances the users requirements” (ISO 8420) and so the durability estimation involves the estimation of the beginning quality preservation during time.

One of the important requirements is the necessity to realize in each building work a balanced link between cost and performances provided; throughout the whole building process, from design and planning, up to construction, use, and demolition. The building has to provide suitable spaces for carrying out activities, which must be comfortable, reliable, able to maintain comfort and safety throughout the time up to the demolition, with a balanced use of resources. The performance’s loss during time, requires maintenance works to restore the programmed conditions, involving costs and resources.

The European Directive 89/106/CEE about the Essential Requirements for Construction Products, and the Interpretative Documents link the expected limit for durability of products and of technical elements realized with them, to the entity of predictable and feasible maintenance works at an economically adequate time.

In particular the Interpretative Documents say that that these requirements have to be satisfied, except for the usual maintenance, for an economically adequate time and define the maintenance as a series of recommendations applied to the building, to satisfy all their functions for the whole service life, defined as the time in which the building performances have to be maintained at a level compatible with the satisfaction of the Essential Requirements.

An economically reasonable service life supposes that all the relevant aspects are considered: planning costs, building and use costs, costs due to the use impediment, risks and damage throughout the building’s service life and the insurance costs covering these risks, costs for programmed renewal in part, costs for inspection, maintenance, supervision and repairing, operation and administrative costs, costs for the demolition and for the environmental aspects.

All this means that the costs- performance balance is the reference for the quality of entire building process: the cost assumes the connotations of global cost, i.e. the costs for construction, management and demolitions, and hence the quality assumes the connotations of global quality that shall be present in all the phases of building process.

If the cost’s concept is associated to the concept of economic and material resources’ use, it’s easy to understand that a qualitative approach in a general extension to the building process can’t ignore the problem of construction’s sustainability.

In particular about the discussed questions, it emerges that building components’ durability, at different levels of object and functional complexity, constitutes one of the factors through which to govern the sustainable consumption of resources.

Coherently with these considerations the durability study of the building’s component for the presents a double purpose: on one side the estimation of the natural service life, i.e. the time in which the expected performances voluntary remain at programmed levels, on the other side the acquisition of the knowledge about a time break and the formalities about which the performance decay is carried on. Natural service life and formalities of performance decay compose the foundations knowledge on which we can arrange the design and the planning of the maintenance operations necessary to the preservation in the time of the programmed use conditions of the building.

In building terms the user’s requirements, the satisfaction of which is the scope of every operation, find their operating expression in the behavior requirements.

One of the fundamental requirements that the users ask for the building systems is to have comfortable and suitable rooms for the foreseen activities: the environmental requirements are the expression of a fundamental need which has to be satisfied through fair environmental performances.

The environmental performances are obtained through the organized contribution of the technological performances, provided by the elements composing the technological system and subsystems.

On the other side these performances provide the answer to the technological requirements which must be satisfied by each specific technological subsystems, operative expression of the users requirements related with the technological system.

The performances of each technological subsystems, are provided by the ability of the component to develop the corresponding simple and complex technological functions. This happens by virtue of specific sets of characteristic properties, with fixed intensities, owned by the materials, sections, elements assumed for the realization of the functional elements setting up the component. These characteristics the components belonging to different subsystems are called functional characteristics and their structured set composes the functional model of the building component. The values, fixed by the design, of the parameters defining the object technical and relation characteristics are called technical specifications.

The maintenance during time of the environmental performances is then strictly linked with the maintenance of the planned intensity of fixed functional characteristics owned by the functional elements, composing the component, the technological performances of which contribute to the realization of the environmental conditions. This means that the functional model of the single component and the pertinent technical specifications have to keep during time coherent with the functional model and with the technical specifications owned at starting time. In other words the technical conformity of the technical elements must be preserved.

The building, and the elements which compose it, are exposed to context conditions, characterized by the environmental agents and the agents produced by the use, developing actions dependant on them.

Usually, we may say that the actions developed during time by the agents produce different kind of effects on the components and on the materials, sections, and elements of which the technical elements are composed, changing the characteristics and the parameters values characterizing them.

For a certain building the characteristics' decay phenomena during time produces an intensity change of the technological performances linked with the environmental performances.

The durability of a component expresses the propensity to supply during time the planned performances through the durability of materials, sections, and elements which compose it, i.e. through the durability of the relevant characteristics.

There are two ways of building components' performances' loss: the first way is characterized by the sudden passage from the condition of good operation to a no operation condition: these components are called bistable. For them the damage or no operation can be univocally identified.

The other way is characterized by a continuous loss of functioning, and so these elements are said not bistable. In this kind of decay there is the necessity to define the level of performance decay acceptable for the identification of the damage condition.

As the technological performances, given by a component belonging to a building in a certain stress context, have the aim to realize some suitable environmental performances for the expected activities, the limits of acceptable decay of a technological performance, which contribute to the realization of a defined environmental performance, is identified when the change of this environmental performance is pointed out and for which the environmental conditions are no more realized at a satisfactory level. It is to notice that the suitable environmental conditions for developing some activities, which satisfy the requirements, change when the social-economic context changes or when the use model changes. From this point of view the definition of damage condition or breakdown is relevant to the specific cultural and physical context.

In the same way the intensity of each component's technological performance is defined by virtue of the intensity of its contribution for the attainment of the planned environmental conditions in the particular arrangement of performance loads that the design has planned for that element.

The kind and the intensity of the technological performances, supplied by the components of the different building' subsystems, can be defined on the base of the answer that a certain technological subsystem must develop in comparison with the kind and the intensity of the stress by the agents present in the context, to take part in the realization of adequate environmental conditions for the expected use. As a consequence the acceptable threshold of the change of a component's technological performance, a change that, as we have said, identifies the beginning of the damage, can't be generalized but it is strictly connected with the intensity of the stress agents present in the specific physical context and with the particular arrangement of the performance loads that the design has given to the component.

All the same the intensity change in the characteristics of materials, sections, and elements composing each building component, because of the specific context agents, takes over a meaning of damage only when the specific technological function, which is developed through it, is no more able to come to the planned intensity to obtain the expected value of the corresponding technological performance. If the durability essentially consists in, for all the elements at a different level of complexity (materials, sections, elements, components, subsystems which realize the building) the ability to preserve for a certain time the characteristic values within acceptable limits, it stands out that the durability with absolute value doesn't exist, as the quality with absolute value doesn't exist. Only values exist related to the building, in a certain context of use and of stress, with the aim to satisfy the requirements which have put into action for that specific operation.

## 2 THE DURABILITY EVALUATION

The canonical method to get indications about building components' service life, the time in which the changes happen, requires the observation during time of technologically homogeneous building objects samples, of which is known the starting date of operation, and subject to stressing homogeneous agents: this observation appears very difficult.

These difficulties, together with the need to test components of which we have not reliable information, puts us the need to start with an adequate experimentation through laboratory tests upon specific samples.

Taking into account the reported considerations, the time behavior of a building has to be governed in the decisional moment of the process through the assignment to the components of a correct propensity to the time behavior. The durability control is carried out during the component's design through the engagement of building products with a controlled decay, coherent with the estimated building service life. For this reason the design must have the knowledge about the decay phenomena and about the life cycles of building parts at different level of complexity.

At the building's level it's necessary to know the trend of the differentiated technological obsolescence of the functional parts composing the building system, due to the different object connotation of the parts themselves and in relation with the specific context stressing conditions. At the component's level it's necessary to know the trend of the differentiated performance decay during time of the different functional parts of the technological system related to each characterizing technological requirement.

As a consequence the durability estimation has to be followed up at the level of each significant functional part of the technological system, i.e. at the level of the component's category. Moreover, for each category the durability estimation is different for each technological performance corresponding to each requirement of the considered class.

In synthesis the time behavior estimation of the technological system must be followed up at the level of components' category, must be tested one by one for each specific technological performance of the specific category, must be connected with the specific environmental and use conditions, that is at the expected stress state for the category use in the specific building.

The durability estimation, to be correctly worked out, requires the measurement of the parameters which makes it operable, that is: the damage rate or instant reliability, the service life reliability, that is the probability of the expected service life , the service life of the "object" also said average time of good operation.

Among these parameters the service life and the reliability are necessary for a component's category durability estimation, which can bring a correct information for the design phase and for the building maintenance planning. Then the durability estimation must be articulated in a service life estimation and in a estimation at the service life reliability.

As the time behavior presents different connotations for the different components, because of the phenomenon of different functional obsolescence, the knowledge of the different durability values allows the design phase to work out and to plan adequate maintenance works, to restore the loss functionality, measured during time with the scope to reach an adequate cost-performance balance.

A correct prediction of works to carry out in the building management process supposes the availability of sufficient knowledge about the damages for the various components, about the expected time of the damage happening (service life or average time up to a damage), about the probability that the expected time of service life can be reached (reliability),as well as about the foresight of the damage happening in the time unit ( damage rate).

A rigorous estimation of the durability of the building components belonging at a certain category consists in the experimental evaluation of their time behaviors in the various environmental conditions and in use conditions in which these elements are stressed.

The difficulty to practice the rigorous estimation and the necessary long time have justified the team of Politecnico di Milano to approach the durability estimation with a methodology which allows to by-pass the above mentioned difficulties.

This methodology is articulated in a method for the service life estimation on the base of the experimental phase applied to components' samples and in a method for the reliability estimation to be applied to the design of components, out of system and with no intended use (Figure 1).

## 3 THE METHODOLOGY FOR SERVICE LIFE ESTIMATION

The methodology for the service life estimation is based on experimentation through accelerated aging laboratory tests and through natural weathering aging tests applied to partial and complete samples of building components of non load-bearing external walls.

The components' decay phenomena are investigated according to the sequence agents-actions-effects. In the climatic test cell of the technological laboratory the agents rain, freeze-thaw, humid hot and dry hot conditions are simulated. The principles for the definition of the stress context and then of the typology and intensity of natural and artificial agents to estimate the decay, the principles for the individuation of the simulated agents with which we can carry out the tests in a technological laboratory, the principles for the set-up of these tests, that is the cycle structure, with the definition of the intensity of the agents, the application time and their sequence in the tests development are presented in Rigamonti 2002.

The laboratory accelerated aging tests, carry out to the individuation and to the quantification of the sensibility of the tested components to the actions by the activated agents, subset of the agents really present in the reference context.

With the term "sensibility" we mean the complex of changes at different level of intensity of the characteristics owned by the objects subject to the test in comparison with the values owned at the beginning time. During aging the functional characteristics of materials constituting the samples, the trend of technological functions developed through those characteristics and then the corresponding technological performances are monitored.

The "aging" tests of the samples are accelerated tests, that is the stress agents, equal as quality to the real ones, are artificially intensified or accelerated in relation to the application time: the time of decay appearing is not real in laboratory. To reach the foresight of the real time before damage, or natural service life, it's necessary to work out a re-scaling, exposing some samples to outdoor natural conditions with actions and stressing agents equal to the ones developed in the technological laboratory. When the measurement of the outdoor components points out an effect already seen in the accelerated tests, it's possible to define a time range to transfer the laboratory accelerated time results, in real terms and it's possible to express the actual service life as real time.

The natural aging tests allow to measure the time when the different intensities of monitored characteristics' changes appear. The correlation between accelerated tests and natural tests for a limited time allows to define a real time scale where to report the measured accelerated effects. It's also necessary to elaborate criteria to define for these effects the damage threshold and the time when the damage occurs, i.e. the service life time.

The research group is working on not bistable components with regards to the way damages appear, i.e. they are characterized by the progressive functions' loss. For them it occurs to define the acceptability threshold of performance decay in virtue of the acceptability threshold of environmental performances. For this reason the research has set up a simulating theoretical method which allows to read contemporary the changes of the environmental performances at standardized values, recognized as adequate to the use model for the residential building users, and referred to a conventional space, with fixed dimension, depending on the changing of technological performances given by the space envelope components, in virtue of the intensity changes of functional characteristics owned by the functional elements which constitutes the component.

In the simulating model this happens through calculation algorithms, to express the technological functions through which the considered technological performances are given. The functional characteristics threshold values, for which the conformity of the environmental performance occurs no more, are called performance limits and are composing the reference term in the technological laboratory to define the achievement of the damage state in test development as reported in Re Cecconi 2002.

## 4 THE METHODOLOGY FOR RELIABILITY PROPENSITY ESTIMATION

Then the service life estimation is worked out on the base of the direct experimental results or methodologically mediated by the time behavior of the component out of system and with no intended use, in conventional stress conditions; on the contrary the reliability estimation at service life time is susceptible to be followed up in terms of propensity through the analysis of the components functioning, i.e. through the functional analysis, of objects and of the structure of the components design.

The method is based on the analysis of the functional model of building components belonging to a defined category, model referred to the designed component and defined through the functional analysis according to Maggi *et al.* 2000.

For each technological subsystem one defines the base functions, corresponding to the connotative technological requirements, and then defines the articulation into analytical functions for the development of the base functions according to Maggi *et al.* 2001.

The functioning model defines a component at the performance specifications phase level, (i.e. the functional model composed by the planned functions) and at the design level, (i.e. the object model composed by objects correlated between them and carrying definite functional characteristics with fixed values of intensity).

The functioning model of a certain component schematizes, through the functional model, the functional structure of the component, (that is the different functional places to which are awarded the analytical functions which bring to the development of the base functions, subset of the technological functions characterizing the subsystem) their position, their mutual relations, and, through the object model, the physical structure of the component itself, that is the different layers or functional elements, their position and their mutual relations.

The characteristics, identified by the kind and by intensity, owned at zero time by the products which compose the functional elements of a component, designed or already realized, are singled out throughout the reading of the object model.

The method for the estimation of the reliability's propensity consists in the application to the functional model and to the object model of the components, belonging to a certain catalogue, of specific judgment principles following four evaluation keys. The reliability 's propensity, as the probability to reach the average time of good operation or, for the other side, as the probability of a quickly damage, finds its origin in the component's design moment for what concerns the functional reliability, inherent and critical and in the construction moment for what concerns the executive reliability.

The estimation of the functional reliability's propensity is worked out on the component's functional model. Usually we observe that the distribution structure of the analytical functions in functional places leads to a greater or smaller risk of performance loss in relation with the stress to which the functional elements are subject. The judgment categories for the estimation of the functional reliability's propensity are: simplicity of the model, stress of the model, distribution of the functions.

The estimation of the executive reliability's propensity stands out by the analysis of the executive complexity of the building component and it's based on the foresight of the possible difference between the built component and the designed one, due to the execution mistakes because of the object complexity of the component. The mistakes which can happen during the construction phase are twice: the first consists in the positioning of the component out of its co-ordination space; the second consists in the set up of materials, sections, units in a way not corresponding to the design indications.

The risk of these mistakes occurring is as bigger as higher is the technological dimensional complexity of the component. The technological dimensional complexity is evaluated from the object model on the base of the judgment criteria for material complexity, object complexity and relation complexity.

The estimation of the inherent reliability's propensity is carried out on the component's design and is based upon the foresight of the activation of the inherent phenomena throughout the component's life. These phenomena in the whole component and in the functional elements composing it, produce dimensional changes resulting more or less not allowed by the kind of joints links between these components. The impediment to the free explication of the "dimensional training" gives stresses which can determine breakage. The stress due to the context, which determines the dimensional changes in exercise, are due in particular to temperature and humidity changes. For them two aspects of the elementary reliability are identified: the thermal inherent reliability and the humid inherent reliability.

The estimation of the critical reliability's propensity is carried out about the project of the component and is based upon the foresight of the possible damages which can occur because of the chemical and chemical-physical incompatibility between the functional elements of the component. These phenomena produce changes of different intensity in the structural integrity of the component operating, from the lightest ones interesting only the component's appearance to the gravest ones which compromise its structural integrity.

## 5 AN EXAMPLE OF THE USE OF THE METHODOLOGY

The proposed methodology has been applied experimentally by the research group of the Department of Engineering of Building Systems and Urban Planning of Politecnico di Milano and by SUPSI (CH) on a non load-bearing external wall named PV2.

The initial technological performances of this component have been evaluated by calculation and the values of the functional characteristics assumed for the evaluation have been defined. (Figure 2).

By simulating the performances' decay the performance threshold limits have been evaluated according to Maggi et al 2000 e to F. Re Cecconi 2002. In figure 1 an example is presented about the decay of performance "Surface condensation control" and the definition of performance limits for the functional characteristics of thermal conductivity.

In the SUPSI Laboratory three sets of component's samples have been built: upon one set the initial characterization testing have been carried out, while on the second set laboratory accelerated aging has been applied in a weathering test cell according to Maggi et al 2001 e to G.Rigamonti 2002. Finally the last samples' set has been exposed to natural outdoor aging in Italy and Switzerland, to be monitored in order to estimate the time re -scaling according to Maggi et al 2000, 2001.

On the PV2 component design it has been evaluated the reliability propensity following the reported methodology according to Maggi et al 2001: the final scores have been presented in figure 2.

## 6 CONCLUSIONS

The presented method, also with different approaches, defines a procedure following which to reach the parameters' estimation for the time behavior evaluation of the building components' of a certain class.

Through the theoretical simulation of space models it allows to define the damage status, i.e. the value of each functional characteristics that generate no more acceptable values of linked environmental performances.

Through accelerated tests in a technological laboratory and natural weathering tests it allows to define the degradation effects and the service life time, i.e. the time when the damage values are reached, with functional characteristics' modeling and related performances.

Through the procedure of reliability propensity estimation it allows to evaluate on the component's design the risk of early functioning loss depending on its functional and object structure.

The results of the application of this methodology supply the building components' design with a valid contribution to the durability control.

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**Figure 1. The methodology for the evaluation of building components' durability**

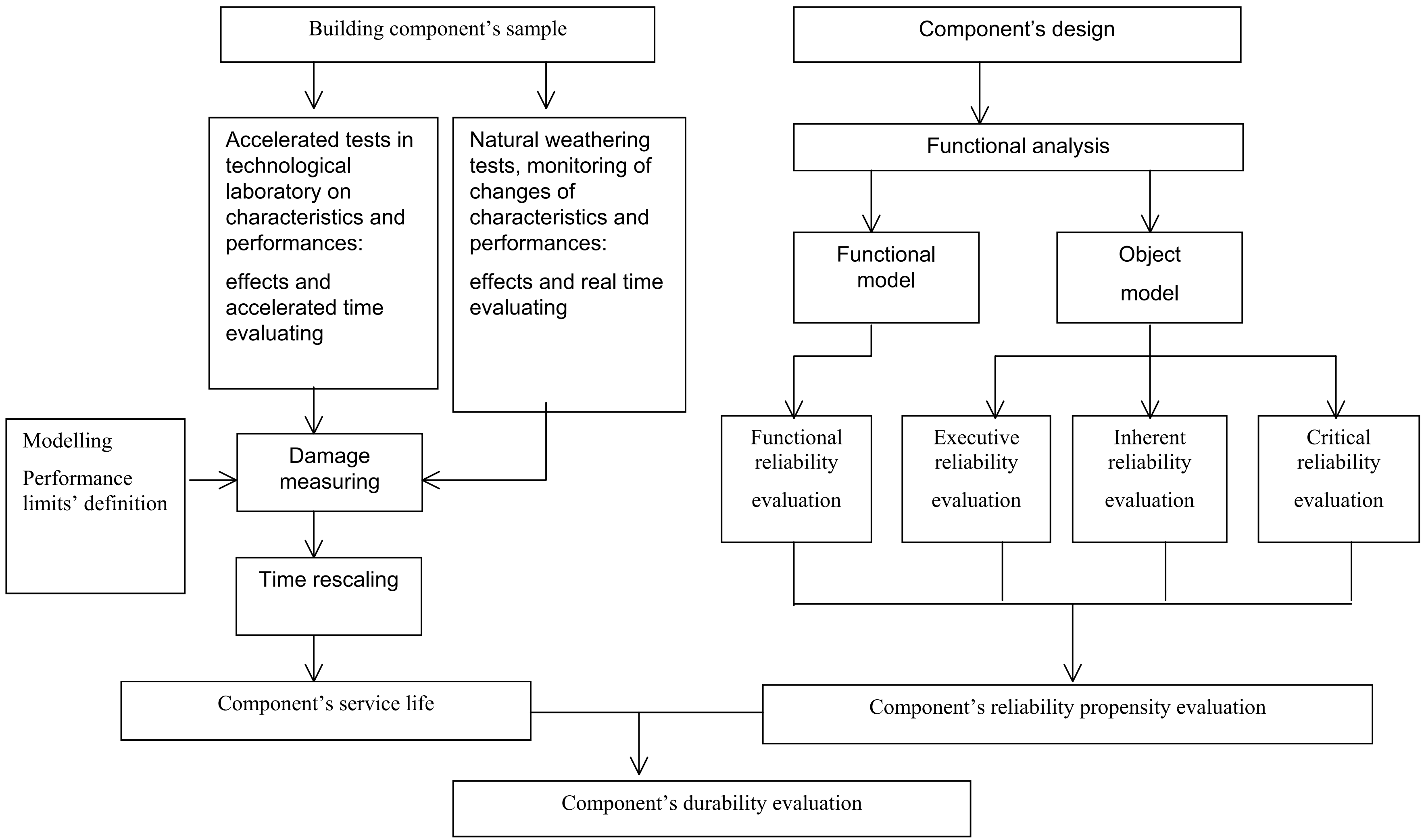

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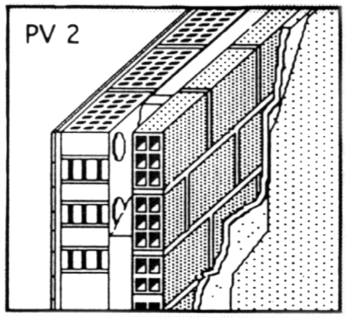
**Figure 2. External non load-bearing wall "hollow and half full bricks" (PV2)**

1 Acrylic resin's thick painting
2 External plaster
3 Half-full bricks
4 Synthetic-cement adhesive
5 Glass fiber panel
6 Hollow bricks
7 Internal plaster
8 Acrylic resin's painting




*Parameters and functional characeristics' values*

| | *Depth* | *Density* | *Thermal conductivity* | *Specific heat* | *Vapor resistance factor* | *TTC* |
|---|---|---|---|---|---|---|
| | *cm* | *kg/m³* | *W/m K* | *kJ/K kg* | *Non dimension* | *h* |
| 1 | 0,2 | 1000,0 | 0,900 | 0,91 | 15 | 1,60 |
| 2 | 1,5 | 1800,0 | 0,900 | 0,91 | 35 | 28,00 |
| 3 | 12,0 | 1070,0 | 0,440 | 0,84 | 7 | 44,80 |
| 5 | 4,0 | 30,0 | 0,036 | 0,97 | 1 | 2,13 |
| 6 | 8,0 | 730,0 | 0,350 | 0,84 | 6 | 25,60 |
| 7 | 1,5 | 1800,0 | 0,900 | 0,91 | 35 | 28,00 |

Technological Performances' Evaluation

| *Technological performance* | *Parameter* | *Symbol* | *Unit* | *Value* |
|---|---|---|---|---|
| Interstitial condensation control | Condensation score | $S_c$ | Non dim | 4 |
| Surface condensation control | Outdoor temperature to activate condensation | $T_{sc}$ | °C | ≤ -15 |
| Winter thermal inertia control | Thermal Time Constant (in>out) | TTC | h | 63,6 |
| Thermal insulation | Thermal transmittance | K | W/m$^2$ K | 0,83 |

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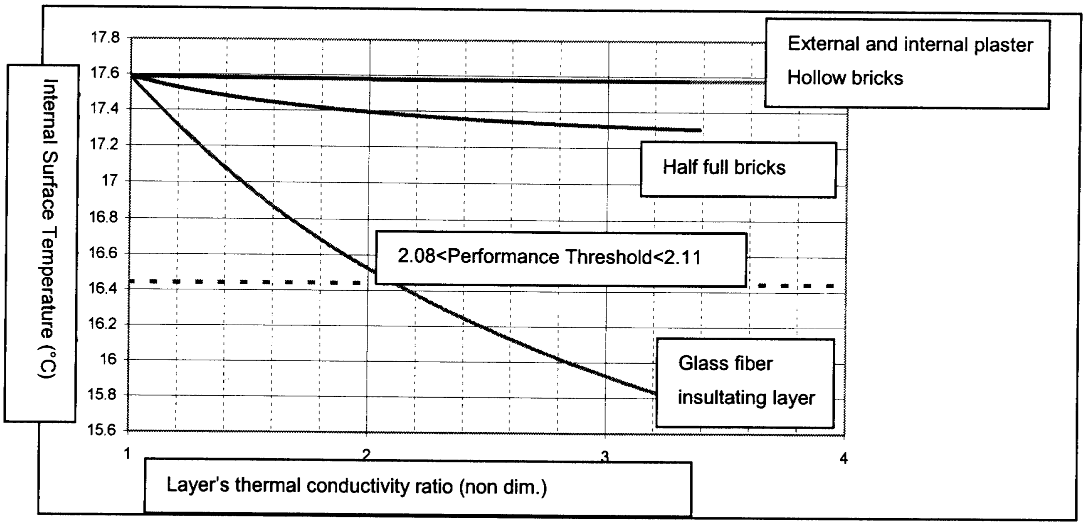


| *Reliability propensity score: Global Reliability* | *2,16* | | |
|---|---|---|---|
| Executive Reliability | 4,24 | Inherent Reliability | 1,73 |
| Functional Reliability | 2,39 | Critical Reliability | 2,63 |

# The Effect Of Fly Ash And Recycled Aggregate On The Corrosion Resistance Of Steel In Cracked Reinforced Concrete

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**Summary**: **Recycled aggregate concrete is an example of a sustainable construction material, especially in the presence of fly ash. This work was aimed to investigate the effect, if any, of recycled aggregate and/or fly ash on corrosion behavior of steel in cracked reinforced concrete. When used, as much fly ash as cement was added in the concrete mixture.**

**Since less alkaline conditions in the concrete pore solution, as those caused by fly ash addition because of its pozzolanic activity, seem to favor a dense protective passive layer on galvanized steel reinforcement, a possible beneficial effect of fly ash addition on the corrosion resistance of galvanized steel in cracked natural or recycled-aggregate concrete was also studied.**

**The results obtained showed that galvanization always improves the corrosion resistance of reinforcements, also in the presence of cracks and in very aggressive chloride environments. The introduction of the sustainability concept in concrete by using recycled aggregate and/or fly ash has no deleterious effect on the corrosion behavior of bare steel reinforcement, since the corrosive attack turns more diffuse and less penetrating, especially when pozzolanic additions are made. As far as galvanized steel reinforcement is concerned, the use of recycled aggregate and/or fly ash seems even to improve its corrosion resistance**

**Keywords. Reinforcement Corrosion, Cracked Concrete, Recycled Aggregate, Fly ash, Galvanized Steel.**

## 1 INTRODUCTION

The manufacture of durable concrete for buildings can contribute to construction sustainable development (Metha 1997) based on principles, techniques and materials that preserve natural resources and improve the environmental quality by the use of recycling in concrete.

Buildings material, that are no longer able to fulfill their original task after a certain time, can be processed in such a way that they can be used again in the original application as recycled aggregate (O'Brien 1998). Replacing natural with recycled aggregate in concrete reduces the consumption of natural resources and protects natural site features. This also avoids the environmental impact resulting from quarries and allows a reduction in the volume of materials disposed to landfill.

The use of fly ash in concrete (Malhotra & Bilodeau 1999) allows to recycle the energy spent in the industrial process, which generates fly-ash as a by-product with the simultaneous reduction of waste disposal to the environment. In particular, due to its pozzolanic activity, fly ash can be used by partially replacing cement, thus reducing the energy consumption and the carbon dioxide emission related to cement production.

Previous experiments (Corinaldesi & Moriconi 2001) showed the possibility to compare the structural properties of recycled aggregate concrete, when fly ash is added, with the ones of ordinary concrete for current applications. However, the structural serviceability, in terms of corrosion tests on reinforced concrete (Hansen 1992), especially in the presence of cracks, which often affect the concrete cover of real structures, has not yet been investigated.

Therefore, the aim of this work was to study if the introduction of the sustainability concept in concrete, by using recycled aggregate and/or fly ash, could influence the corrosion behavior of cracked reinforced concrete.

Moreover, from a holistic point of view, sustainable construction means also to design a reinforced concrete structure with the appropriate durability (Ellis jr & Shelton 1995) in relation to the specified service life and, when it is exposed to particularly aggressive environments, additional reinforcement protection, such as galvanization (Swamy 1990, Yeomans 1994, Fratesi, Moriconi & Coppola 1996) has to be used to reduce repair costs and environmental loads. Since galvanization protection seems to be favored by the alkalinity reduction (Andrade *et al.* 1983) of the concrete pore solution which can be achieved by fly ash addition because of its pozzolanic activity, in this work the corrosion resistance of galvanized steel in a so-called sustainable cracked concrete, containing fly ash and/or recycled aggregate concrete, was also studied.

## 2 EXPERIMENTAL

Concrete prism specimens (280 mm × 70 mm × 70 mm) were manufactured with 6 different concrete mixtures (Table 1), with natural or recycled aggregates, in the presence or absence of fly ash;

- w/c = 0.3 with natural aggregate and without fly ash (N0.3)
- w/c = 0.6 with natural aggregate and fly ash (N0.3F)
- w/c = 0.6 with natural aggregate and without fly ash (N0.6)
- w/c = 0.3 with recycled aggregate and without fly ash (R0.3)
- w/c = 1.2 with natural aggregate and fly ash (N0.6F)
- w/c = 0.6 with recycled aggregate and fly ash (R0.3F)

When present, a low calcium fly ash (ASTM C 618 Class F), was added as a cement replacement at the dosage rate of 50% by cement weight.

The six different concrete mixtures were deliberately chosen in order to classify the manufactured concretes into three different strength grades respectively equal to 50-60 MPa, 30-40 MPa and 10-20 MPa.

N0.3 concrete belongs to the highest strength class as a reference high quality concrete.

Ordinary concretes generally used with compressive strength of about 30-40 MPa are represented by N0.6 concrete. N0.3F and R0.3 concretes belong to this strength class, because in the presence of fly ash or recycled aggregate the water to cementitious material ratio has to be lowered to 0.3 in order to achieve the same strength class value.

Finally, poor quality concretes with compressive strength of about 10-20 MPa, are represented by R0.3F and N0.6F concretes.

**Table 1. Concrete mixtures proportions.**

| Mixture | w/c | Mixture Proportions, kg/m$^3$ | | | | | | w/cm | Compressive Strength (MPa) |
|---|---|---|---|---|---|---|---|---|---|
| | | Water | Cement | Sand | Crushed Aggregate | Fly Ash | Super-plasticizer | | |
| N0.3 | 0.3 | 165 | 550 | 317 | 1350 | - | 5.5 | 0.3 | 58 |
| N0.3F | 0.6 | 165 | 275 | 298 | 1269 | 275 | 5.5 | 0.3 | 38 |
| N0.6 | 0.6 | 230 | 380 | 314 | 1338 | - | - | 0.6 | 31 |
| N0.6F | 1.2 | 230 | 190 | 301 | 1282 | 190 | - | 0.6 | 16 |
| R0.3 | 0.3 | 165 | 550 | 372 | 1060 | - | 5.5 | 0.3 | 33 |
| R0.3F | 0.6 | 165 | 275 | 330 | 991 | 275 | 5.5 | 0.3 | 21 |

A commercial portland-limestone blended cement, Type CE II/A-L 42.5 R according to the European Standards EN-197/1, was used.

Crushed limestone aggregate (15 mm maximum size) and natural sand (6 mm maximum size) were used to make the reference natural-aggregate concrete. A coarse fraction (15 mm maximum size) and a fine fraction (6 mm maximum size) were also used to make concrete containing recycled aggregate. These fractions were directly supplied by an industrial crushing plant in Villa Musone, Italy, in which rubble from demolition of buildings are cleaned, crushed and sized.

When proportioning the concrete, the two above-mentioned kinds of aggregates - recycled or natural - were used with the same gradation, obtained by combining the fine and the coarse sieve fractions, according to the Bolomey particle size distribution curve. Moreover, the mixture workability in relation to the particles shape, as well as the cement dosage, were taken into account in order to optimize the gradation. Since no detection of the actual relative humidity of the aggregates was carried out, the amount of mixing water was always adjusted to obtain the same fresh workability of 150-180 mm slump.

A 30% aqueous solution of an acrylic-based admixture was used a superplasticizer to compensate for the workability loss caused by the addition of low calcium fly ash (ASTM C 618 Class F) and to allow the use of w/cm as low as 0.3.

Each prismatic specimen was reinforced with a steel plate (210 mm × 40 mm × 1 mm), either bare or galvanized, embedded with 3 cm concrete cover. The zinc coating on the galvanized steel plate was 100 µm thick, obtained by molten zinc immersion, with an outer pure zinc layer about 20 µm thick. The galvanized steel plates, just before embedment in the fresh concrete, were submerged for 5 sec in 15% NaOH solution to dissolve the $ZnCO_3$ layer eventually formed during atmospheric storage. The electric contacts between the reinforcing plates and the measurement equipment were arranged according to previously reported methodology (Fratesi, Moriconi & Coppola 1996).

After 1 month of air curing the specimens were cracked by flexural stress so that a crack width of 1 mm was produced in a pre-formed notch area with the apex crack reaching the reinforcement. Then the specimens were exposed to weekly wet-dry cycles (2 days dry followed by 5 days wet) in a 10% NaCl solution.

The corrosion risk of the reinforcement in the concrete specimens exposed to the aggressive environment was evaluated by free corrosion potential measurements with a saturated calomel electrode (SCE) as reference, while the kinetic of the corrosion process was monitored by polarization measurements. The polarization resistance was measured through the galvanodynamic method, using an external graphite bar as counter-electrode, by calculating its average value between the anodic and cathodic ones.

The electrochemical values reported in the following graphs are averaged among the measurements carried out on three specimens of each type during the full immersion period.

In order to validate and to complete the evaluation of the electrochemical behavior, the concrete specimens were autopsied after 7 wet-dry cycles in the chloride solution to assess the corrosion by visual observation. All the steel plates were removed after splitting the concrete specimens. The weight loss and the surface of the corroded area on bare steel plates was evaluated after pickling, while metallographic analysis were carried out on the cross section of the galvanized steel plates to evaluate the coating thickness decrease due to the corrosive attack. Zinc corrosion products were identified by X-ray diffraction and the free chloride concentration on the reinforcements was also measured at the end of the test by water extraction.

## 3 RESULTS AND DISCUSSION

### 3.1 Bare steel

#### 3.1.1 Electrochemical measurements

Figure 1 shows the free corrosion potential values of bare steel plates embedded in cracked concrete as a function of wet-dry cycles.

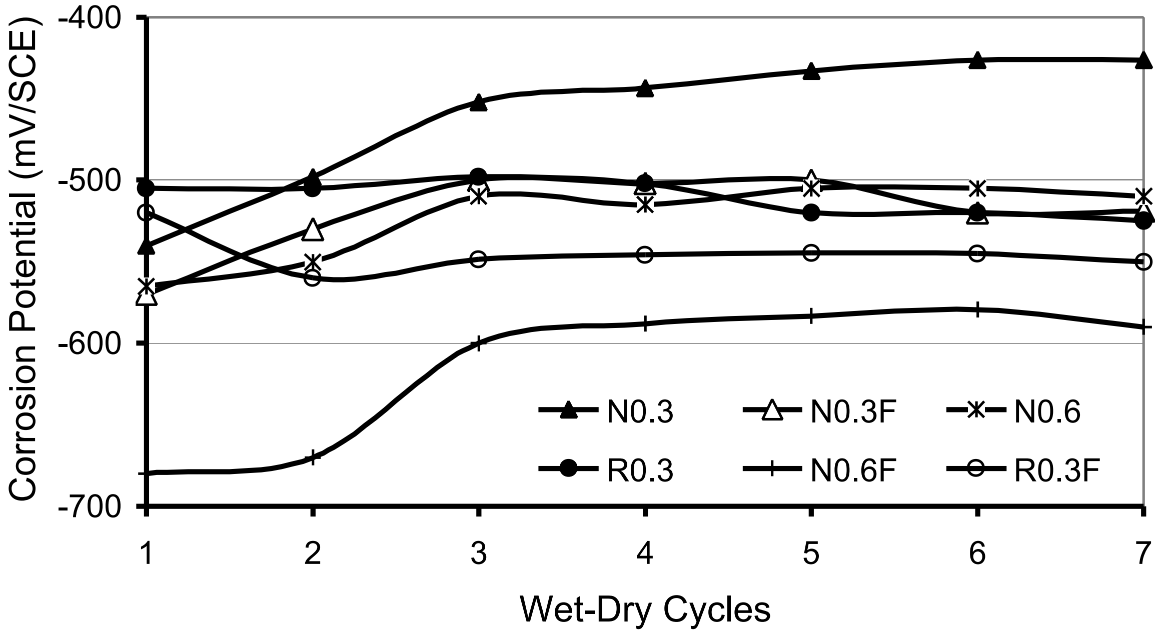


**Figure 1. Corrosion potential of bare steel plates embedded in cracked concrete as a function of wet-dry cycles.**

It can be observed that just after the exposure to the chloride environment, all the steel plates assumed activation values lower than –500 mV/SCE, regardless of the cement matrix type, reflecting a general high corrosion risk. However, after few wet-dry cycles, the steel plates embedded in the most porous concretes (N0.6F and R0.3F) showed the highest corrosion risk, with the lowest corrosion potentials values. On the contrary, the steel plates embedded in the reference high quality concrete (0.3N) showed the lowest corrosion risk. Finally, the steel plates embedded in the other concrete mixtures belonging to the ordinary class strength (30-40 MPa) assumed corrosion potential values converging to an intermediate corrosion risk.

The corrosion behavior described by the corrosion potential measurements were completely confirmed by the polarization resistance measurements (Fig. 2). Also the corrosion resistance of the steel reinforcement seemed to depend mainly on the mechanical strength of the cracked concrete, regardless of the cementitious matrix type. In this case, obviously, the higher the concrete strength class is, higher the corresponding polarization resistance turns.

Therefore, the opportunity of manufacturing eco-compatible concrete, by replacing natural with recycled aggregate and/or by adding high fly ash volume, does not seem to negatively affect the corrosion behavior of the embedded steel reinforcements as far as the same strength class is concerned. Indeed, in ordinary concrete with compressive strength of about 30 MPa, recycled-aggregate concretes and high volume fly ash concretes show a higher polarization resistance. Moreover, the results obtained could dispel any doubt on the uncertain corrosion behavior of steel in high volume fly ash concrete, due to the concrete pore solution alkalinity reduction caused by the fly ash pozzolanic activity, at least in the presence of cracks in concrete.

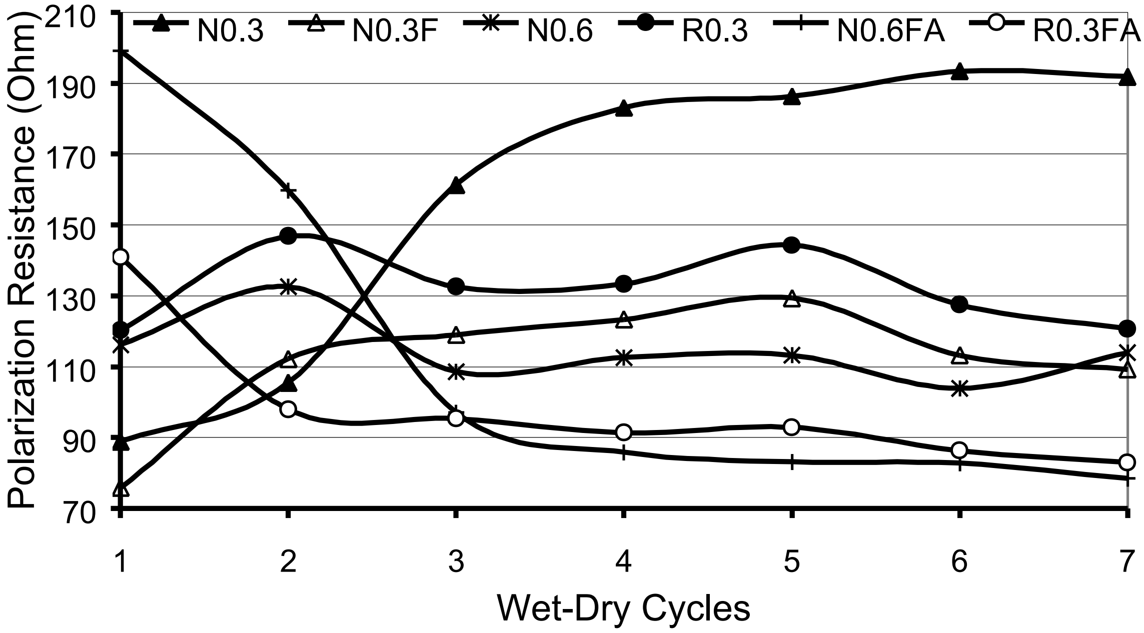


**Figure 2. Polarization resistance of bare steel plates embedded in cracked concrete as a function of wet-dry cycles.**

3.1.2 Autoptic evaluation

The electrochemical tests represent a partial feature of the reinforcement corrosion behavior since instantaneous measurements can only be carried out during the wet period and there is no chance to estimate the metal surface area effectively corroding. Therefore, after 7 wet-dry cycles all the specimens were split in order to visually evaluate the corroded area and to assess the weight loss of the bare steel plates after pickling.

All the reinforcements were characterized by a strong corrosive attack in the crack area which could be justified by the high chloride concentration detected on the reinforcement surface (2-5% by cement weight) significantly greater than the concentration threshold (0.4% by cement weight) generally reported as the critical value able to induce the corrosion process in bare steel.

However, from a morphological point of view (Fig. 3), in the presence of recycled aggregate and, particularly, of fly ash the corrosive attack appeared more diffuse and less penetrating with respect to that detected on the steel plate embedded in the reference concrete, where a more localized corrosive attack could also induce the steel plate blanking in spite of the very low w/c.

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**Figure 3. Visual observation of the corrosive attack on the bare steel plates embedded in reference natural-aggregate concrete N0.3 (left), in recycled-aggregate concrete R0.3 (middle) and high volume fly ash concrete N0.3F (right).**

At last, the bare steel plates weight loss turned out quantitatively in good agreement with the electrochemical measurements (Table 2).

**Table 2. Weight loss of the bare steel plates due to the corrosive attack.**

| MIXTURE | WEIGHT LOSS (gr) |
|---|---|
| N0.3 | 0.28 |
| N0.3F | 0.30 |
| N0.6 | 0.38 |
| R0.3 | 0.38 |
| N0.6F | 0.36 |
| R0.3F | 0.47 |

### 3.2 Galvanized Steel

3.2.1 Electrochemical measurements

Figure 4 shows the free corrosion potential values of galvanized steel plates embedded in cracked concrete as a function of wet-dry cycles. It can be observed that just after the exposure to the aggressive environment, the corrosion potential of the galvanized steel plates assumed initial active values of about –1000 mV/SCE regardless of the concrete type. During the wet-dry cycles, also in the case of galvanized steel reinforcement, a higher corrosion risk represented by low potential values was recorded for poor quality concrete (N0.6F and R0.3F). The reference high quality concrete (0.3N) assumed the highest potential values and the ordinary concretes (R0.3, N0.6 N0.3F) showed intermediate values, indicating the lowest and intermediate corrosion risk respectively.

The polarization resistance measurements carried out on galvanized steel reinforcements (Fig. 5) again turn in good agreement with the potentials values as for bare steel, by confirming the already stated hierarchy related to the concrete strength class.

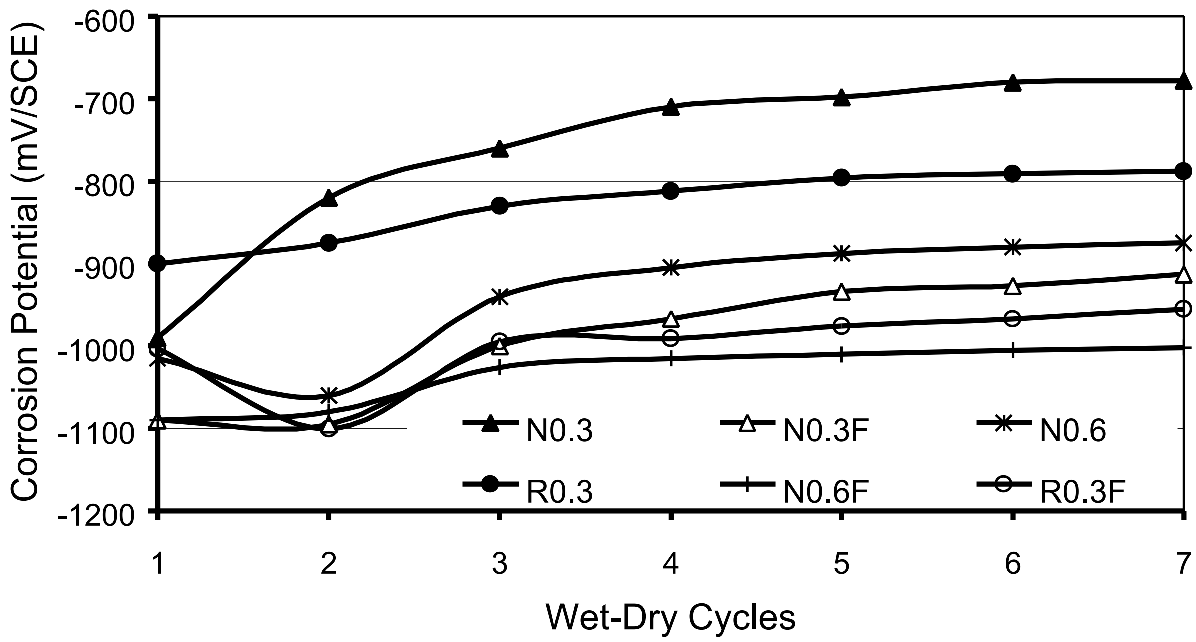



**Figure 4. Corrosion potential of galvanized steel plates embedded in cracked concrete as a function of wet-dry cycles.**

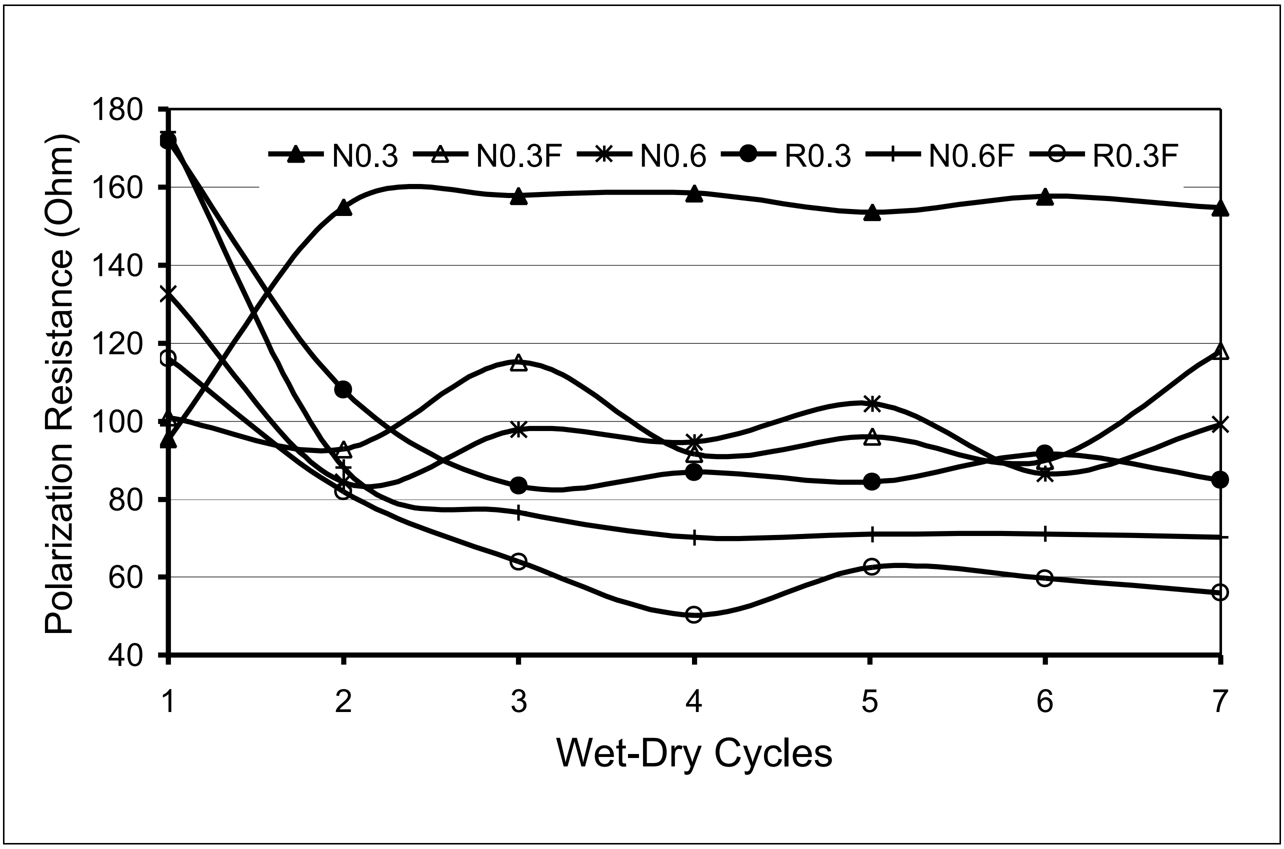
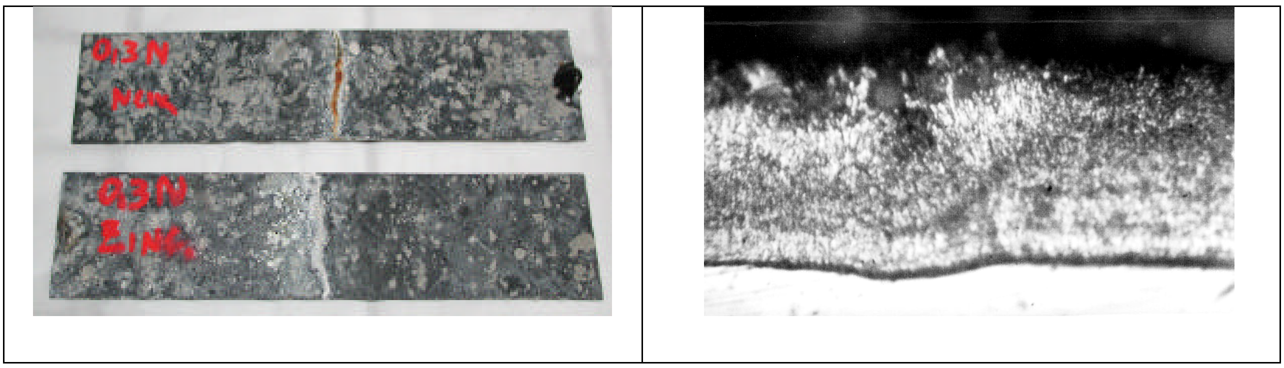



**Figure 5. Polarization resistance of galvanized steel plates embedded in cracked concrete as a function of wet-dry cycles.**

3.2.2 Autoptic evaluation

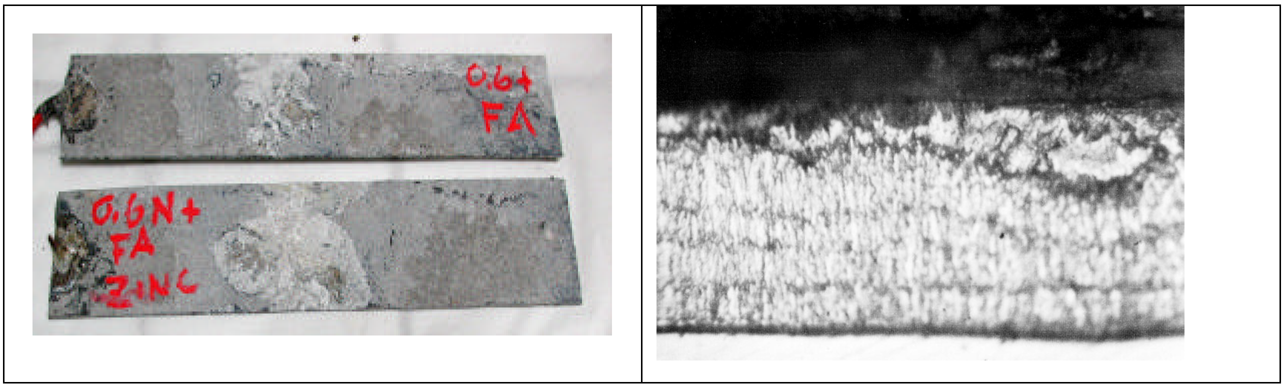
The autopsy carried out in this case revealed additional unexpected information with respect to that obtained by the electrochemical measurements.

As a matter of fact, in the reference high quality concrete (0.3N), where the reinforcement polarization resistance was high, the corrosive attack really appeared very localized at the crack apex and also very deep, even showing well recognizable iron corrosion products (Fig. 6a). Moreover, far from the crack apex, Fe-Zn alloy appeared on the plate surface meaning that total consumption of the pure zinc layer due to the corrosive attack occurred, as later confirmed by metallographic analysis (Fig. 6b).



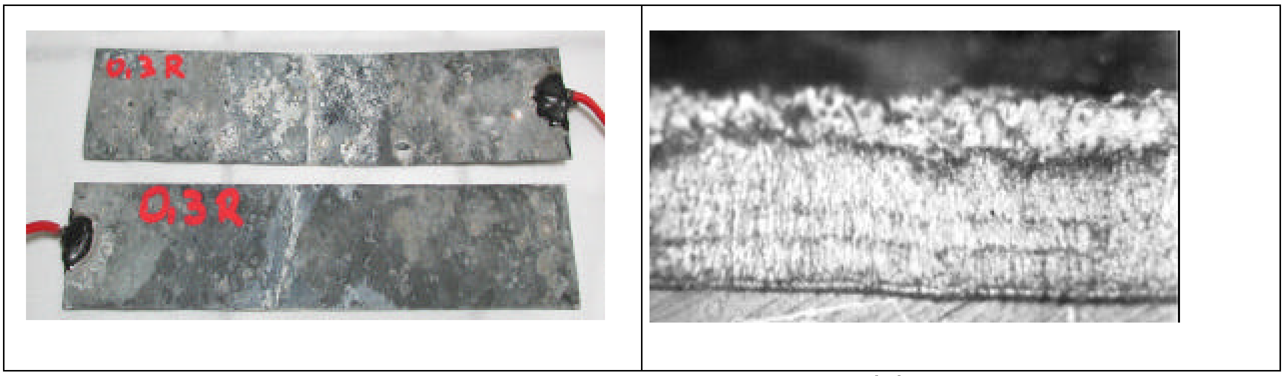

(a) (b)

**Figure 6. Visual observation (a) and metallographic cross section (b) of the galvanized steel plates embedded in reference concrete (0.3N).**




**(a)** **(b)**

**Figure 7. Visual observation (a) and metallographic cross section (b) of galvanized steel plates embedded in high volume fly ash concrete (0.6NF).**




**(a)** **(b)**

**Figure 8. Visual observation (a) and metallographic cross section (b) of galvanized steel plates embedded in recycled-aggregate concrete (0.3R).**

On the other hand, the galvanized steel plates extracted from high volume fly ash concrete showed a surface coating made of zinc corrosion products (Fig. 7a), later identified as calcium hydroxyzincate by X-ray diffraction. Calcium hydroxyzincate is a well passivating zinc corrosion product (Duval & Arliguie 1974, Macias & Andrade 1987) whose formation seems to be effectively favored by low alkalinity of the concrete pore solution (Andrade *et al.* 1983) as achieved when high volume fly ash is added because of its pozzolanic activity. Once formed, calcium hydroxyzincate protects the underlying pure zinc layer from further corrosion as metallographic analysis, later carried out on the cross section of the galvanized plates embedded in high volume fly ash concrete, well put into evidence (Fig. 7b).

Unexpectedly, the galvanized steel plates extracted from the cracked recycled-aggregate concrete specimens showed the lowest corrosive attack. Zinc grains were still well visible on the galvanized plates surface after the aggressive exposure (Fig. 8a) and the metallographic observation revealed a continuously thick pure zinc layer still present on the reinforcements (Fig. 8b).

## 4 CONCLUSIONS

Steel galvanization has been shown to improve the corrosion resistance of reinforcement embedded in concrete, also in very aggressive chloride environment and in the presence of cracks in concrete.

The introduction of the sustainability concept in concrete by using recycled aggregate and/or fly ash has no deleterious effect on the corrosion behavior of bare steel reinforcements for similar concrete strength class. Indeed, especially when fly ash is added, the morphology of the corrosive attack turns more diffuse and less penetrating.

The use of high volume fly ash in concrete significantly affects the corrosion behavior of galvanized reinforcement embedded in cracked concrete by generally improving its corrosion resistance.

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# Durability Of Recycled-Aggregate Concrete Incorporating High Volumes Of Fly Ash

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**Summary: The results presented in this paper are part of a research program dealing with the use of recycled concrete as an aggregate to manufacture structural concrete. Emphasis is placed on durability related properties rather than engineering properties. Durability is assessed in terms of carbonation and chloride penetration depth as well as freezing and thawing resistance.**

**Reference concrete was manufactured by using only natural aggregates with a water/cement of 0.60 as commonly used in practice. Then, a recycled-aggregate concrete was prepared by completely substituting natural aggregate, with a water/cement of 0.30 in order to achieve the same strength class of the reference concrete by a safety margin. This recycled-aggregate concrete was also manufactured by using a water-reducing admixture in order to lower both water and cement dosage, without disregarding the prefixed strength class requirements. Moreover, a recycled-aggregate concrete was manufactured with a water/cement of 0.60, by simultaneously adding to the mixture as much fly ash as cement, substituting the fine aggregate fraction. Thus, a water to cementitious material ratio of 0.30 was obtained enabling the concrete to reach the prefixed strength class. This procedure is essential for designing an environmentally friendly concrete. All concretes were prepared maintaining the same fluid consistency by proper addition of an acrylic-based superplasticizer.**

**Compressive strength was measured at 3, 7, 28 and 56 days; the 28-day compressive strength always fell in the range of 27-31 MPa.**

**Carbonation and chloride penetration tests were carried out to compare the four concretes in terms of durability behavior. In general, this durability seems to be better guaranteed by recycled-aggregate instead of reference concrete, mainly owing to the lower water to binder ratio.**

**The resistance against freezing and thawing cycles was investigated on concrete specimens prepared without any air-entraining admixture, in order to verify the influence of recycled instead of natural aggregates. No change in the freezing and thawing behavior attributable to this replacement was detected.**

**Keywords. Recycled-aggregate concrete, High volume fly ash, Carbonation, Chloride penetration, Freeze-thaw resistance.**

## 1. INTRODUCTION

A judicious use of natural resources, which are rapidly depleting, represents one of the main action meeting the concept of sustainable development, the other being an overall reduction of environmental impact. The former action can be achieved by re-using industrial by-products thus reducing both material waste and natural materials consumption. Recycled-aggregate concrete containing a high volume of fly ash is an example of a construction material that is in harmony with this concept. In fact, in this concrete rubble from building demolition can be introduced, after a suitable treatment, as a natural aggregate fraction substitution. In this way the volume of rubble disposed to landfill could decrease and simultaneously also natural resources consumption could be reduced. Moreover, the addition of fly ash, which is a by-product of thermal power generation, due to its pozzolanic activity, allows to reduce cement dosage and, as a consequence, to reduce carbon dioxide emissions into atmosphere, causing the well-known “greenhouse” effect, related to cement production.

This work is a part of a wide research program related to the use of recycled aggregate to manufacture structural concrete: a recycled aggregate coming from a real industrial crushing plant, therefore readily available to employ, was used by completely substituting in concrete both fine and coarse natural aggregate fractions. The feasibility of manufacturing structural concretes by using this aggregate was already successfully tested (Corinaldesi *et al.* 1999), then the influence of fly ash addition on mechanical performances was evaluated, again with positive results (Corinaldesi & Moriconi, 2001). Finally, also some aspects about durability of recycled-aggregate concretes were studied; in particular attention was focused on the influence of concrete porosity on drying shrinkage and corrosion of embedded galvanized steel bars (Corinaldesi *et al.* 2001). Results showed that, when fly ash was added to recycled-aggregate concrete:

1. the pore structure was improved, and particularly the macro pores volume was reduced causing benefits in terms of mechanical performance;
2. from the serviceability point of view the drying shrinkage of recycled aggregate concrete did not appear a problem since, just due to less stiffness of this concrete, the same risk of cracks formation resulted as for the ordinary concrete;
3. as far as corrosion aspects were concerned, the use of fly ash appeared very effective in protecting galvanized steel reinforcement also in porous concrete, as it could occur when recycled aggregates are used, even in the case of cracked concrete.

## 2. SCOPE

The main objectives of this study were to investigate the following aspects concerning durability of recycled-aggregate concretes, eventually containing high volumes of fly ash:

- resistance to carbon dioxide penetration, that is concrete carbonation;
- resistance to chloride ions penetration into concrete;
- resistance to freeze-thaw cycles of concrete prepared without any air-entraining admixture.

In any case, the behavior of an ordinary concrete prepared with natural aggregate and commonly used in practice was also estimated, as a reference for the performances of recycled-aggregate concretes.

## 3. EXPERIMENTAL PROGRAM

### 1.1 Materials

#### 1.1.1 Portland Cement

A commercial portland-limestone blended cement type CEM II/A-L 42.5 R according to the European Standards EN-197/1 was used. The Blaine fineness of cement was 4.15 $m^2/g$ and its specific gravity was 3.05 $kg/m^3$.

#### 1.1.2 Natural Aggregates

Crushed limestone aggregate (15 mm maximum size) and natural sand (6 mm maximum size) were used. The gradation of both crushed aggregate and sand are shown in Fig. 1 and their physical properties are shown in Table 1.

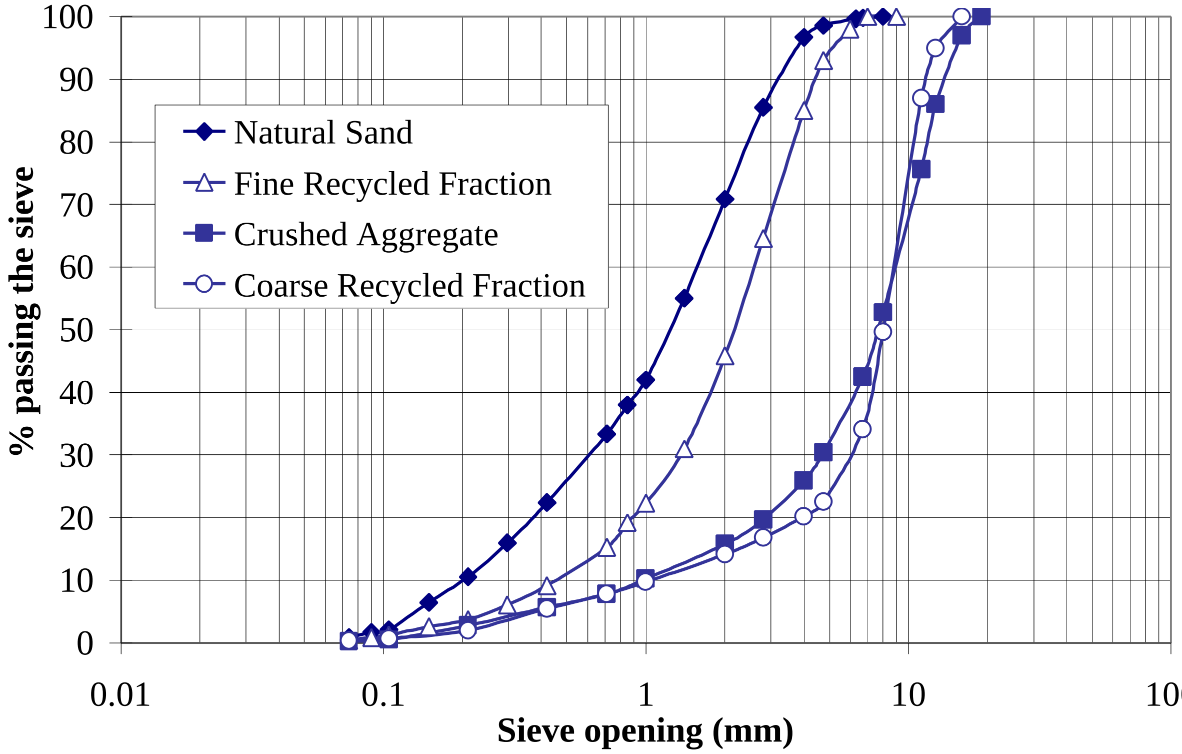


**Figure 1. Grain size distribution curves of the aggregate fractions.**

1.1.3 Recycled Aggregates

A coarse fraction (15 mm maximum size) and a fine fraction (6 mm maximum size) of recycled aggregate were used. These fractions were directly supplied by an industrial crushing plant in Villa Musone, Italy, in which debris from building demolition were suitably selected, ground, cleaned and sieved. The gradations of both coarse and fine recycled aggregate are shown in Fig. 1 and their physical properties are shown in Table 1.

**Table 1. Some physical properties of the aggregate fractions**

| *Aggregate Fractions* | *Bulk Specific Gravity* (kg/m$^3$) | *Water Absorption* (%) | *Passing 75 mm* (%) |
|---|---|---|---|
| Natural Sand | 2620 | 3 | 0.9 |
| Crushed Aggregate | 2680 | 2 | 0.2 |
| Fine Recycled Fraction | 2150 | 10 | 0.5 |
| Coarse Recycled Fraction | 2320 | 8 | 0.3 |

1.1.4 Superplasticizer

A 30% aqueous solution of acrylic-based superplasticizer was used when required to guarantee fresh workability.

1.1.5 Fly Ash

A low calcium fly ash (ASTM C 618 Class F) produced by a thermal power generating station was used. The Blaine fineness of fly ash was 4.48 m$^2$/g and its specific gravity was 2.25 kg/m$^3$.

The chemical compositions of portland cement and fly ash are reported in Table 2.

**Table 2. Chemical composition of cement and fly ash**

| *Oxide* (%) | *Cement* | *Fly Ash* |
|---|---|---|
| $SiO_2$ | 29.67 | 59.94 |
| $Al_2O_3$ | 3.74 | 22.87 |
| $Fe_2O_3$ | 1.80 | 4.67 |
| $TiO_2$ | 0.09 | 0.94 |
| CaO | 59.25 | 3.08 |
| MgO | 1.15 | 1.55 |
| $SO_3$ | 3.25 | 0.35 |
| $K_2O$ | 0.79 | 2.19 |
| $Na_2O$ | 0.26 | 0.62 |
| L.O.I. | 11.62 | 3.34 |

**1.2 Concrete mixtures**

The mixtures proportions are reported in Table 3.

**Table 3. Concrete mixture proportions**

| *Mixture* | *NAT-0.6* | *REC-0.3* | *REC-WRA-0.3* | *REC-FA-0.6* |
|---|---|---|---|---|
| W/C | 0.60 | 0.30 | 0.30 | 0.60 |
| W/CM | 0.60 | 0.30 | 0.30 | 0.30 |
| *Mixture proportions, kg/m³* | | | | |
| Water | 230 | 230 | 165 | 230 |
| Cement | 380 | 760 | 550 | 380 |
| Fly Ash | - | - | - | 380 |
| Natural Sand | 314 | - | - | - |
| Crushed Aggregate | 1338 | - | - | - |
| Fine Recycled Fraction | - | - | 372 | - |
| Coarse Recycled Fraction | - | 1169 | 1060 | 1057 |
| Superplasticizer | - | - | 5.5 | 6.8 |

Concrete specimens were prepared by using two different kinds of aggregate, recycled or natural, of the same maximum diameter (15 mm) and the same sieve distribution, obtained by combining a fine and a coarse sieve fraction according to the Bolomey particle size distribution curve (Collepardi 1991) and assuming the physical properties values reported in Table 1 for each kind of aggregate. Moreover, the mixture workability in relation to higher or lower aggregate irregularity and cement dosage was taken into account.

Reference concrete was manufactured by using only natural aggregates with a water to cement ratio of 0.60 as commonly used in practice (NAT-0.6).

Then, a recycled-aggregate concrete was prepared, by completely substituting natural with recycled aggregate, with a w/c of 0.30 (REC-0.30) in order to achieve the same strength class of the reference concrete. However, in this way, a very high cement dosage was necessary to obtain the strength requirement and this occurrence could partially compromise the environmental benefit related to the use of the recycled instead of natural aggregate.

Therefore, this recycled-aggregate concrete was also manufactured by using a water-reducing admixture (WRA) in order to lower both water and cement dosage (REC-WRA-0.3): actually, an acrylic-based superplasticizer was used at a dosage of 1.0 % by weight of cement.

Moreover, a recycled-aggregate concrete was prepared with a w/c of 0.60, by simultaneously adding to the mixture as much fly ash (FA) as cement, to replace the fine aggregate fraction (REC-FA-0.6); however, in this way a water to cementitious material ratio of 0.30 was obtained.

Fresh concrete had a preferred fluid consistency (slump between 16 and 18 cm). Since no detection of the actual relative humidity of the aggregates was carried out, the amount of mixing water was adjusted to obtain such a prefixed slump range. In the presence of fly ash a dosage of 1.8 % by weight of cement of an acrylic-based superplasticizer was also necessary to achieve the prefixed workability.

### 1.3 Casting and curing of test specimens

Cubic specimens, 100 mm in size, were manufactured according to Italian Standards UNI 6130-72 Part I. These specimens were used for both mechanical and durability tests.

The cubic specimens were cast in polystyrene forms and wet cured at 20°C (UNI 6130-72 Part II).

## 4. RESULTS AND DISCUSSION

### 1.4 Compressive strength

Compression tests according to Italian Standards UNI 6132-72 were carried out on cubic specimens, which were tested at right angles to the position of casting. Therefore the bearing faces were sufficiently planar and smooth as to require no capping or grinding. The specimens were loaded at a constant strain rate until failure.

For each mixture and each curing time (3, 7, 28 and 56 days) three specimens were used for mechanical tests, according to Italian Standards.

Figure 2 shows the compressive strength as a function of time for the four concretes prepared. It can be noticed that after 28 days of curing the compressive strength was in the range of 27-31 MPa.

It can be also observed that at early ages the w/c is more influent than the kind of aggregate on the strength development.

Moreover, particularly effective pozzolanic activity is showed by fly ash in the presence of recycled aggregate since early ages. However, at longer ages, all the observed strength developments tend to converge. This means that the use of a weaker aggregate, as the recycled one, has more and more influence on the compressive strength as the cement matrix gets stronger, this being the “weak chain link” case (Corinaldesi *et al.* 1999).

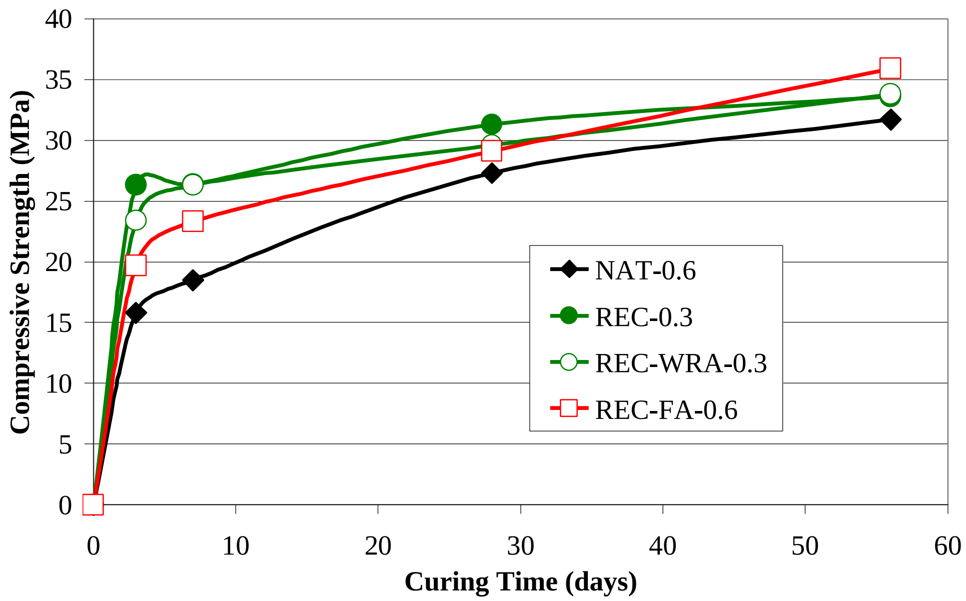


**Figure 2. Compressive strength of different concretes as a function of curing time.**

### 1.5 Depth of Carbonation

The carbonation depth was measured by the phenolphthalein test (RILEM CPC – 18) in cubic concrete specimens exposed to air at a temperature of 20°C after demoulding at 1 day.

Figure 3 shows the carbonation depth (*x*, in mm) as a function of the time of exposure to air (*t*, in days). A linear relationship between the carbonation depth and the square root of time ( $x = k \cdot \sqrt{t}$ ) can be drawn; the extrapolated values of *x* after 1 year were 8.5, 6.1, 5.2 and 2.1 mm respectively for the concretes NAT-0.6, REC-FA-0.6, REC-WRA-0.3 and REC-0.3.

It is quite evident that the carbonation depth is mainly influenced by the water/cement and particularly effective it seems to be a high dosage of cement (see the mixture REC-0.3 in Table 3) and consequently a high amount of available calcium hydroxide. Moreover, also the positive contribution of fly ash addition is detectable (comparing the results obtained for NAT-0.6 and REC-FA-0.6) mainly due to the refinement of the pore system.

A similar experimental procedure was conducted on high volume fly ash concretes prepared with natural aggregate and a water to binder ratio of 0.32 (Collepardi S. *et al.* 2000). The results of this experimental program showed that the carbonation depth was un-detectable or negligible after some months of exposure to air.

In general, it is confirmed that carbonation does not pose problems for corrosion of the metallic reinforcements, due to very low permeability of concretes containing high volumes of fly ash (Sivasundaram *et al.* 1990), also in this case, though a porous aggregate, as the recycled aggregate, was present.

Figure 4 shows the pictures taken some minutes after the phenolphthalein test, relative to specimens NAT-0.6 and REC-0.3, previously kept in exposure to air for 130 days.

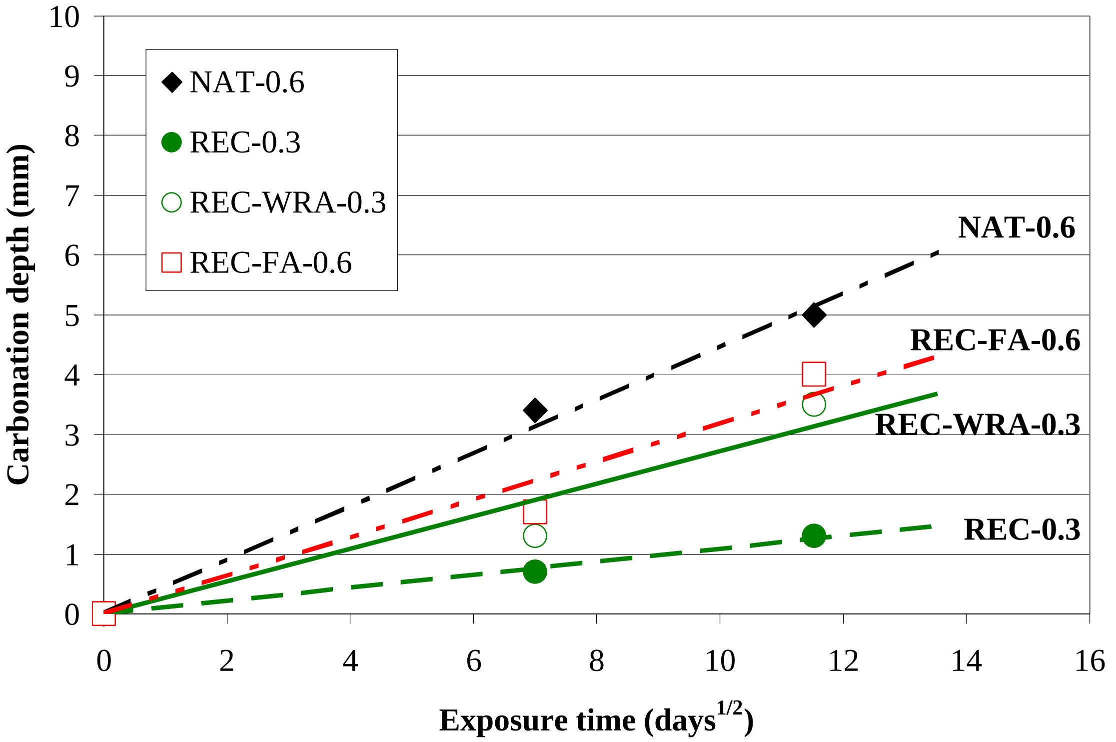


**Figure 3. Carbonation depth as a function of the time of exposure to air.**

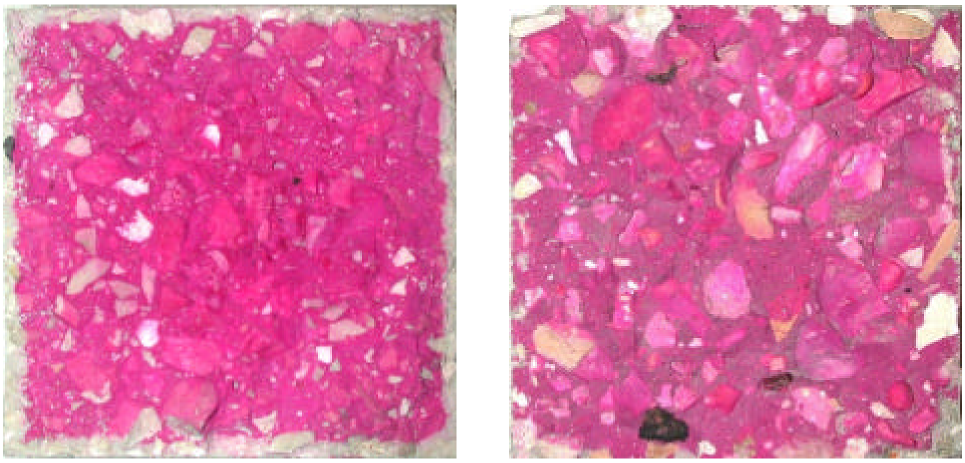

**Figure 4. Carbonation depth after 130 days of exposure for NAT-0.6 (left) and REC-0.3 (right) concretes**

### 1.6 Chloride penetration

The chloride penetration into concrete was evaluated through the $AgNO_3$ and fluorescein test (Collepardi *et al.* 1972) in concrete specimens exposed to a 10% NaCl aqueous solution after a wet curing of 1 week and an air curing of 3 weeks at a temperature of 20°C.

Figure 5 shows the chloride penetration depth as a function of the time of exposure to a 10% NaCl aqueous solution.

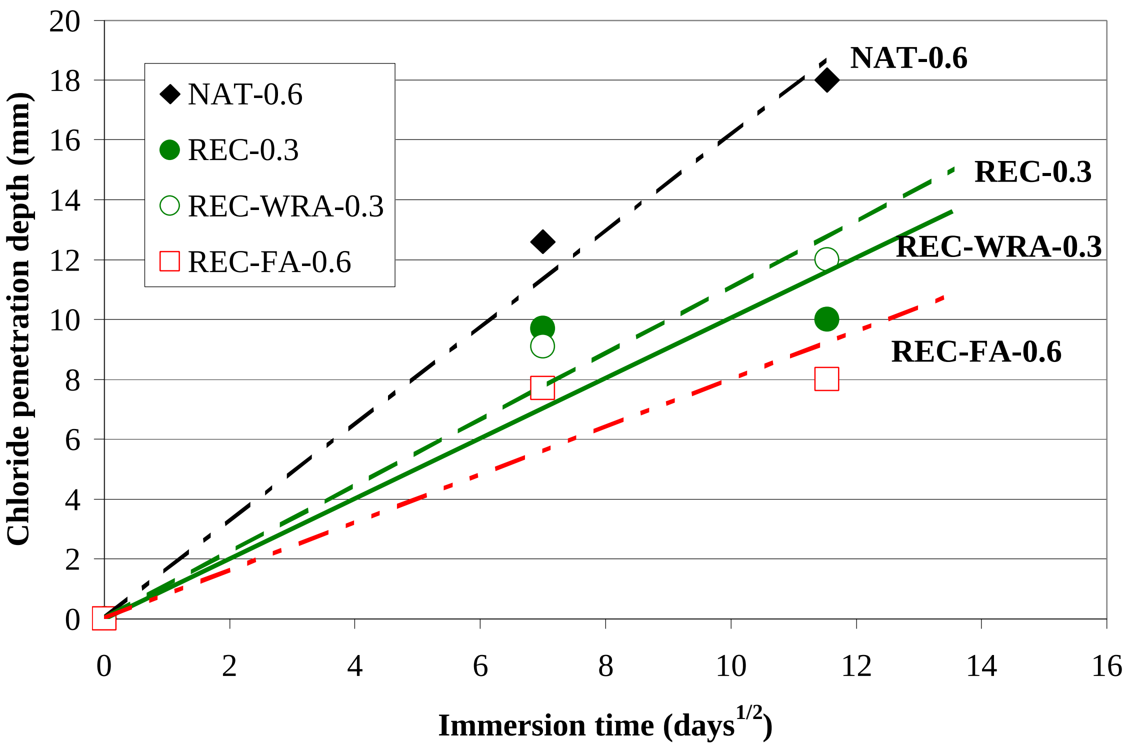


**Figure 5. Chloride penetration as a function of the time of exposure to a 10% NaCl aqueous solution.**

Collepardi *et al.* (1972) found that chloride penetration depth ($x$) varies with elapsed time ($t$) following an equation [1] obtainable from the solution of Fick's second law under non-steady-state conditions for diffusion in a semi-finite solid:

$$x = 4 \cdot \sqrt{D \cdot t} \qquad [1]$$

where $D$ is the diffusion coefficient of chloride ions into concrete pores filled of water, expressed in $cm^2 \cdot s^{-1} \cdot 10^{-8}$. The values of $D$, obtained from [1] by interpolating the results showed in Fig. 5 for the different concretes, are reported in Table 4.

**Table 4. Diffusion coefficients of chloride ions into different concretes at 20°C**

| *Mixture* | *NAT-0.6* | *REC-0.3* | *REC-0.3+* | *REC-FA-0.6* |
|---|---|---|---|---|
| $D$ ($cm^2 \cdot s^{-1} \cdot 10^{-8}$) | 1.90 | 0.87 | 0.72 | 0.46 |

It is quite evident the strong beneficial effect due to the presence of fly ash on the chloride penetration depth measured for REC-FA-0.6 concrete: in fact the diffusion coefficient of chloride ions into REC-0.3 concrete is double with respect to REC-FA-0.6, the only difference being the replacement of 50% of cement with fly ash.

In the same paper (Collepardi *et al.* 1972), experimental results are reported supporting the hypothesis that the differing nature of the surface of the pores formed in the presence of pozzolanic materials plays a vital role in affecting the behavior of the material during the penetration of chloride ions.

Also a lower water/cement influences positively the concrete resistance against chloride penetration: as a matter of fact, the diffusion coefficient of chloride ions into NAT-0.6 concrete is more than double with respect to REC-0.3, although recycled instead of natural aggregate was present.

Figure 6 shows the pictures taken some minutes after the $AgNO_3$ and fluorescein test, relative to specimens NAT-0.6 and REC-FA-0.6, previously kept in exposure to a 10% NaCl aqueous solution for 130 days.

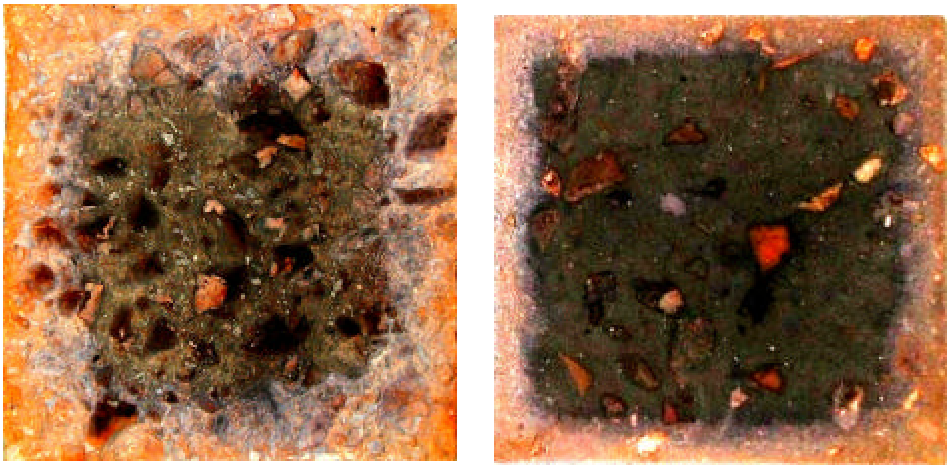

**Figure 6. Chloride penetration after 130 days of exposure for NAT-0.6 (left) and REC-FA-0.6 (right) concretes.**

## 1.7 Frost resistance

The frost resistance was measured according to the “Resistance of Concrete to Rapid Freezing and Thawing” Standard Test Method (ASTM C 666) following the “Procedure B: Rapid Freezing in Air and Thawing in Water”. The freeze-thaw cycle consisted of freeze in air, thaw in water (-18°C to +5°C), and four cycles in 24 hours.

Dynamic elastic modulus values and weights were monitored on the concrete specimens at intervals throughout the test, in order to evaluate both the Relative Dynamic Modulus (RDM) of elasticity and the Percentage of Weight Change (PWC), which are usually the main criteria that indicate the extent of damage of concrete subjected to repeated cycles of freezing and thawing.

The dynamic modulus of elasticity was determined by ultrasonic pulse velocity measurements in saturated concrete specimens. This method is based on the fact that the velocity of an ultrasonic pulse in a material (in this case having a frequency of 50 kHz), $V$, is related to its dynamic elastic modulus, $E_d$, and it is expressed by

$$V = k \cdot \sqrt{\frac{E_d}{r}} \quad \text{with} \quad k = \sqrt{\frac{(1-n)}{(1+n)\cdot(1-2n)}} \qquad [2]$$

where $V$ corresponds to the pulse velocity in m/s, $r$ to the density of the material and $k$ to a constant depending on Poisson's ratio that was set to 0.20 for all the specimens. The pulse velocity was obtained by dividing the path length (100 mm) by the elapsed time (measured in µs). A coupling medium, such as grease, was used to ensure good contact between the transducers and the specimen, since the apparent travel time is affected by the smoothness of the contact surface.

Relative dynamic modulus

The relative dynamic modulus of elasticity, RDM, was calculated as follows

$$RDM = \left(\frac{V_c^2}{V_o^2}\right)\cdot 100 \qquad [3]$$

where $V_o$ is the pulse velocity measured at zero cycles, in m/s, $c$ is the number of cycles at the time of testing and $V_c$ is the pulse velocity measured after c cycles of freezing and thawing.

The results obtained up to 240 cycles are reported in Fig. 7; a significant trend towards low values of RDM was not yet detected in any case, at least up to this experimental time.

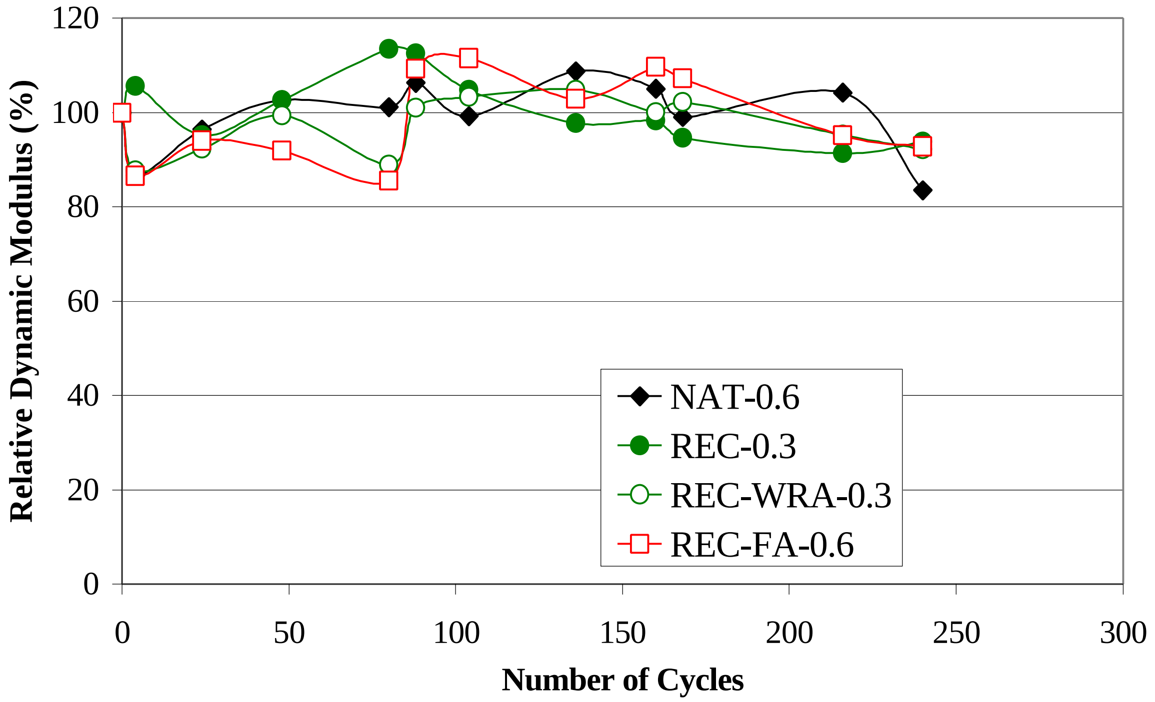


**Figure 7. Relative elastic modulus of elasticity for the different concretes.**

### 1.7.1 Percentage of weight change

The change in the weight of concrete specimens was monitored and the values obtained, expressed in percentage (PWC) with respect to the initial saturated condition, are reported in Fig. 8. The weight of the four concretes increased with the number of cycles raising up to 90 cycles; then a continuos slow decrease, more evident for NAT-0.6 and REC-0.3 concretes, occurred. This may be explained by a water absorption in the first period and by the observed scaling in the second one, which overshadowed the weight increase caused by the ingress of water.

A similar study was carried out by Rohi and Burdette (1998): they found that the use of a higher fly ash content in both natural and recycled-aggregate concrete resulted in significant improvements in their freeze-thawing durability.

In this case, other data must be collected in order to understand the different concrete behavior under freezing and thawing; nevertheless at this moment no negative effect caused by the presence of a more porous aggregate, as the recycled, was detected.

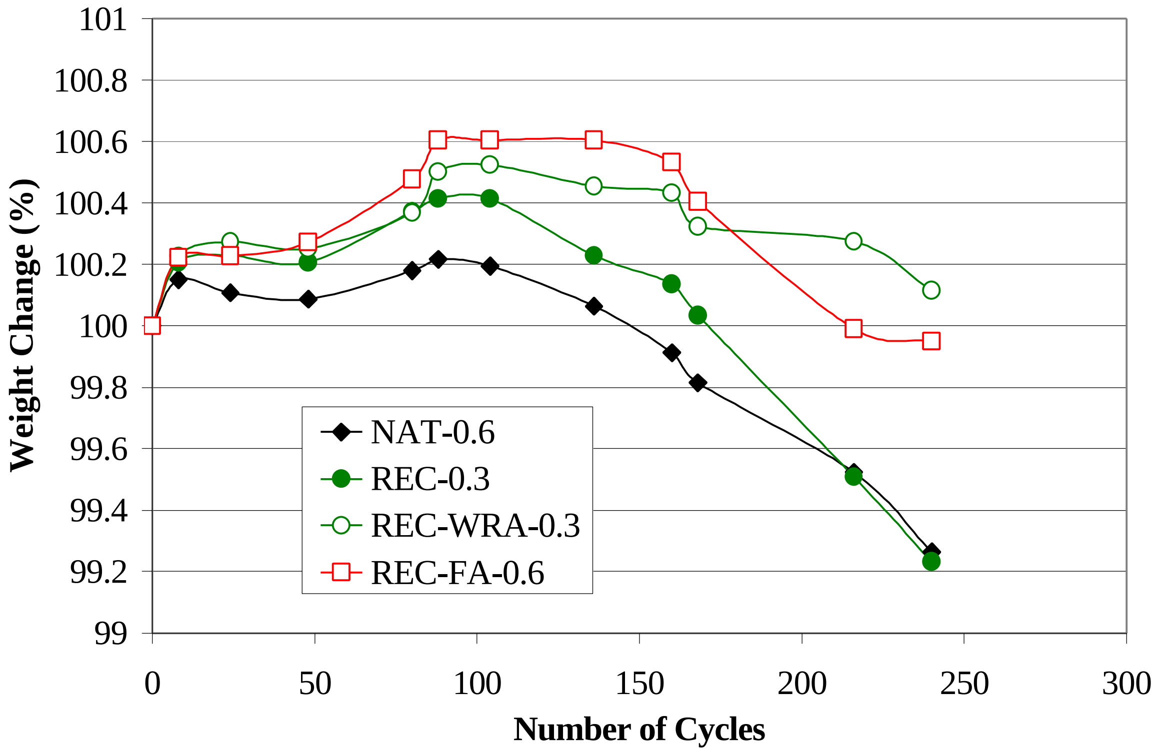


**Figure 8. Percentage of the weight change (PWC) for the different concretes.**

## 5. CONCLUSIONS

The results obtained in this work show that, when the same concrete strength class is guaranteed, not only the use of an aggregate coming from a recycling process is not deleterious for concrete durability but in some cases it could also be useful for durability improvement.

As a matter of fact, due to a lower w/cm, all recycled-aggregate concretes performed better than reference concrete concerning both concrete carbonation and chloride ions penetration into concrete.

Moreover, concerning chloride ions penetration into concrete, a further strong positive effect was related to the presence of high volume of fly ash.

Finally, the test of concrete resistance against freezing and thawing cycles showed no difference between natural-aggregate concrete and recycled-aggregate concretes, at least during the first 240 cycles.

In conclusion, the opportunity of manufacturing an environmentally friendly concrete with satisfactory performances, related to both mechanical and durability aspects, seems not so far to become reality.

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# Accelerated Degradation Testing Of Concrete In Acidic Environment: Resistance To Lactic And Sulfuric Acid

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**Summary: This research was initiated because of durability problems related to building materials in agricultural constructions. Resistance against corrosion caused by lactic and acetic acids is of major importance both for floors in animal houses and silos. Furthermore, concrete manure tanks, walls of manure pits in animal houses and the underside of concrete slats and slabs are exposed to biogenic sulfuric acid corrosion.**

**To examine the degradation of concrete by aggressive liquids, a testing apparatus for accelerated degradation tests (TAP) was developed. It was conceived to obtain acceleration of the deterioration process through alternate wetting and drying, because this procedure simulates the real-life situations under investigation. This is achieved by an experimental set-up with concrete cylinders that rotate through containers with the test solutions. Cycles of chemical attack are followed by abrasion using rotary brushes. The change in dimension of the concrete specimens and the surface roughness are determined through a non-contact distance measurement with laser sensors.**

**In a first experiment the durability of concrete exposed to a solution with 30 g/l of both lactic and acetic acid was examined (pH=2.0-2.2). The effect of the aggregate type, the cement type and the influence of polymer modification (polymer (d.m.)/cement ratio = 0.10) were considered. The results showed that concrete with blastfurnace slag cement was more resistant than the reference concrete with ordinary Portland cement. Use of limestone sand (and aggregates) led to a quicker neutralization of the aggressive liquid, but this was insufficient to reduce the average attack depth. The polymer modifications, especially the use of styrene acrylic ester latex, increased the concrete resistance significantly, although SEM-investigation showed that the emulsified polymer had not completely formed a film.**

**Secondly the resistance to a 0.5% sulfuric acid solution was considered (pH=1.0). Here it was necessary to measure twice per attack cycle: after the alternated wetting and drying stage to determine concrete expansion and after brushing to quantify the decrease in radius due to mechanical action. Compared to the reference concrete with high sulfate resistant Portland cement, durability could be improved by addition of styrene acrylic ester polymer, as well as by using blastfurnace slag cement. Modification with vinylcopolymer had little effect, and addition of silica fume, styrene butadiene or acrylic polymers led in this case to a reduced resistance.**

**Keywords: concrete degradation, lactic acid, sulfuric acid, polymer, silica fume**

## 1 INTRODUCTION

### 1.1 Concrete degradation in agricultural constructions

Various concrete building components in agricultural constructions are subject to acid attack, the main aggressive agents being lactic, acetic and sulfuric acid.

*Concrete floors in animal houses*

Floor types in animal houses can be classified into solid, unperforated floors that are laid directly on the ground and floors that are suspended above ground and may be perforated to assist in the drainage of liquids and passage of fecal material. For solid as well as for suspended floors, those constructed of concrete are the most common. Results from a survey, carried out by De Belie (1997a) on farms with fattening pigs, showed that even for high-quality precast concrete slats, on 15% of the farms surveyed, the coarse aggregates of some slats were exposed within two years of use. After five years, wear was observed on 40% of the farms. Consequences were an increased gap between slat beams and increased surface roughness (resulting in animal injuries), corrosion of the reinforcement and a reduced slat stability. The reason for this degradation are the specific aggressive conditions occurring on floors in animal houses. Chemical components from feed residues and manure may attack the concrete floor surface. Animals and cleaning exert a mechanical impact. Pressures used for high-pressure cleaning can amount to 80-150 bar (Frénay & Zilverberg 1993). Different authors (Hoeksma 1988; Mathiasson *et al.* 1991; Nilsson 1993; De Belie *et al.* 1996a) mention the presence of large amounts of lactic and acetic acid, and aggressive ions such as $SO_4^{2-}$. The major amount of aggressive ions and acetic acid would come from the manure. Lactic acid originates from acidified meal/water mixtures (De Belie *et al.* 1996a) and is the main source of severe concrete degradation near feed and water supply. Acidified meal/water mixtures can reach acid concentrations of 31.0 mg lactic acid and 3.1 mg acetic acid per ml filtrate. The pH can drop to 3.8, in correspondence with the $pK_a$ of lactic acid (3.86 at 25°C). Some farmers add whey to the drinking water and dairy cows and pigs can be fed with silage, which again contains lactic and acetic acids.

*Silage storage structures*

Silage storage structures are used to conserve fodder under anaerobic conditions. Depending on the dry matter content of the crop, the silo, and the silo drainage system, more or less silage effluent is produced, with possibly a pH as low as 3.5-4.0, the main acids being again lactic and acetic acid. On many farms, silage is stored in bunker silos with reinforced concrete floors (and walls), which are readily attacked by the corrosive effluent. Furthermore, wear (abrasion and impact) is caused by farm machinery such as block cutters.

*Manure storage structures*

In 1945, it was found (Parker 1945), that the formation of sulfuric acid in sewers was not due to purely chemical transformations in the sewer atmosphere but to the action of aerobic sulfur oxidizing bacteria, mainly of the *Thiobacillus* species. Anaerobic sulfate reducing bacteria, such as *Desulfovibrio* species, living in manure or in the mud at the bottom of sewer pipes and in the slime at the sides, reduce sulfates and other oxidized sulfur compounds to hydrogen sulfide ($H_2S$). This hydrogen sulfide is released in the air above the liquid level and dissolves in moisture on the walls and on the underside of the slatted floors. There, $H_2S$ can be oxidized to sulfur, a reaction that is catalytically accelerated by the high alkalinity of the concrete surface. The *Thiobacillus* bacteria oxidize the sulfur to sulfuric acid, using the energy released for their cell mass production. Very low pH values (1-2) may occur locally (Mori *et al.* 1991).

### 1.2 Reaction mechanisms

Lactic and acetic acid are very aggressive, because their reaction with free lime [$Ca(OH)_2$] of the concrete produces very soluble calcium salts (Kleinlogel 1960). When those salts are leached, the hydrates of the cement matrix start decomposing and the concrete disintegrates. Sulfuric acid first reacts with the calcium hydroxide in the concrete to form gypsum. Although the formation of gypsum is associated with an increase in volume by a factor of 1.2 to 2.2 (Attiogbe & Rizkalla 1988; De Ceukelaire 1989; Wafa 1994), the reaction between gypsum and calcium aluminate ($C_3A$) with the formation of ettringite is much more detrimental. Some authors mention an increase of volume with a factor two (Attiogbe & Rizkalla 1988; Wafa 1994), while others mention even a factor 7 (De Ceukelaire 1989). Thus the formation of ettringite is mainly responsible for the large volume expansion, which leads to increase of internal pressure and deterioration of the concrete matrix.

## 2 MATERIALS AND METHODS

### 2.1 Aggressive Liquid

For the first experiment the simulation liquid consisted of lactic and acetic acid in water both in concentrations of 30 g/*l*, the highest concentrations registered on floors in animal houses during the preliminary investigations (De Belie *et al*., 1996a). This liquid had a pH of 2.0-2.2, which is extremely aggressive to concrete. The lower pH compared with the pH measured for instance in soured meal-water mixtures with the same acid concentrations, is due to the absence of buffering feed ingredients, and will in reality only occur under specific circumstances. In previous experiments also highly to moderately aggressive liquids were used, with pH-values of 3.8 and 4.5, but it appeared that the three liquids resulted in a very similar classification of the concrete types (De Belie et al. 1996b; 1997a). For the second experiment a 0.5% sulfuric acid solution (pH of around 1.0) was used.

### 2.2 Apparatus for accelerated degradation tests

At the Magnel Laboratory for Concrete Research of Ghent University, an apparatus for accelerated degradation testing (TAP) was developed (Fig.1). A more detailed description of the used test method is given by De Belie (1997c).

**Fig. 1 Apparatus for accelerated degradation testing (TAP)**

It was designed to achieve acceleration of the deterioration process through alternate wetting and drying, because this procedure simulates the real-life situations under investigation. Three cylinders (∅ 270 mm, h = 70 mm) of each concrete mixture were subjected to a cyclic procedure of immersion in an acidic solution and drying by air. The cylinders, fixed on horizontal axes, turned with a speed of 1 revolution per hour trough separate recipients. Each point of the outer circumference was submersed during 1/3 of the rotation time. After each cycle, which lasted for 6 days for the lactic/acetic acid attack and for 12 days for the sulfuric acid attack, the cylinders were dried in air and brushed with rotary brushes to remove weakly adhering concrete particles. In Table 1 the different steps in the test procedure are related to different stages occurring during the degradation in practice.

**Table 1. Relation between different steps in the TAP test procedure and real-life situations**

| *TAP* | *Lactic/acetic acid attack*<br>*(a) concrete floors in animal houses*<br>*(b) silage storage structures* | *Sulfuric acid attack*<br>*(a) sewers*<br>*(b) manure storage structures* |
|---|---|---|
| Cyclic immersion and chemical attack | (a) wetting by feed+water residues and manure, drying in between<br>(b) wetting during effluent release from silage | (a & b) wetting/drying by fluctuation of the wastewater/manure level |
| Drying in air | (a&b) drying during periods the compartment or silo is not in use | (a) dry weather situation with low wastewater level<br>(b) emptying of manure storage facility |
| Abrasion by brushing | (a) abrasion by animals, cleaning with brushes or high-pressure hose<br>(b) abrasion by animals (in self-feeding silo), cleaning and farm machinery (block cutters) | (a) high flow rates because of high loading or rain; abrasion by flowing water, turbulence<br>(b) abrasion by cleaning (less common) |

The corrosion of the specimens was measured using laser sensors, connected with a computer. The sensor amplifiers supply a Volt signal, which is linearly related to the distance between the concrete surface and the sensor. Five measurements per mm are taken along the concrete surface. A software trigger is programmed to start the measurement of a cylinder profile when the raised edge of a stainless angle steel, fixed on the concrete cylinder, passes by the laser beam. The first 50 measurements of a contour line are performed on the horizontal part of the angle steel, which acts as reference plate. The three sensor heads are mounted on a mechanical device which can always be placed in the same position on the frame of the TAP. This device allows to adjust the position of the sensor heads in a direction parallel to the cylinder axes in steps of 0.5 mm with an accuracy of 0.01 mm. In this way it is possible to scan the circumference of a cylinder every time at the same position along the cylinder height. By moving the system a few times, for every cylinder several parallel profiles can be measured, equally distributed along the cylinder height. For the experiments discussed here, 5 (sulfuric acid attack) to 7 profiles (lactic/acetic acid attack) per cylinder were measured. The laser measurements were also used to calculate the surface roughness of the concrete, expressed by means of the $R_a$-value (BS1134 1972).

In the lactic/acetic acid test procedure the radius of the cylinders was determined at the beginning of the experiment and after each cycle of alternated immersion, drying and brushing. In the sulfuric acid test procedure the measurements were performed twice per cycle, before as well as after brushing. In this way it was possible to determine the average change of the radius of the cylinders due to chemical reaction of the concrete with the sulfuric acid solution during the immersion as well as the change of the radius due to mechanical action of brushing the cylinders. The change in radius during the immersion stage corresponded mostly to an expansion of the concrete due to the formation of voluminous reaction products. However, a decrease of the radius could also occur during the period of immersion because of loss of adhesion of the expanded parts.

## 2.3 Test Specimens

The composition of the concrete mixes tested in experiment 1 with lactic/acetic acid is shown in table 2. The composition of the concrete mixes tested in experiment 2 with sulfuric acid is shown in table 3. From each concrete mix three cylinders and six cubes with sides 158 mm were made. The test specimens for experiment 1 were stored for 28 days at 20 ± 2 °C and a relative humidity of 90-95%. The test specimens for experiment 2 were moist cured for 1 day (>90% humidity, 20°C), followed by 3 days of dry curing conditions (60% humidity, 20°C) and subsequent air curing. At 28 days three cubes were measured for compressive strength as prescribed by the standard NBN B15-220 (1990) and the three others were submitted to a water absorption test, which gives an indication of the porosity, as prescribed by the standard NBN B15-215 (1989). The three cylinders of each type were mounted together on one side of an axle of the TAP for the degradation tests.

*Experiment 1: lactic/acetic acid attack*

**Table 2. Composition and characteristics of the fresh and hardened (28 days old) concrete used in experiment 1**

| | *Concrete mix* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Ref I* | *III/A* | *LS-G* | *LS-L* | *SBR1* | *SBR2* | *A* | *SAE* |
| CEM I 42.5R (kg) | 375 | - | 375 | 375 | 375 | 375 | 375 | 375 |
| CEM III/A 42.5 (kg) | - | 375 | - | - | - | - | - | - |
| Sand (kg) | 700 | 700 | 350 | 350 | 700 | 700 | 700 | 700 |
| Lime sand (kg) | - | - | 350 | 350 | | | | |
| Gravel (kg) | 1170 | 1170 | 1170 | - | 1170 | 1170 | 1170 | 1170 |
| Limestone (kg) | - | - | - | 1170 | - | - | - | - |
| Latex* (kg) | - | - | - | - | 81.6 | 75 | 75 | 75 |
| d.m. in latex (%) | | | | | 46 | 47 | 47 | 51 |
| Polymer(d.m.)/cement (%) | | | | | 10.0 | 9.4 | 9.4 | 10.2 |
| Water** (kg) | 146 | 146 | 146 | 155 | 128.5 | 110.7 | 150.7 | 107.7 |
| W/C** (-) | 0.39 | 0.39 | 0.39 | 0.41 | 0.34 | 0.30 | 0.40 | 0.29 |
| Slump (mm) | 5 | 8 | 0 | 3 | 130 | 105 | - | 25 |
| Flow (-) | 1.32 | 1.18 | 1.08 | 1.02 | 1.93 | 1.89 | - | 1.32 |
| Fresh density (kg/m$^3$) | 2400 | 2385 | 2395 | 2420 | 2271 | 2286 | 2310 | 2375 |
| Compressive strength (N/mm$^2$) | 52.0 | 51.9 | 51.7 | 54.1 | 34.7 | 37.2 | 28.6 | 51.2 |
| Water absorption (%) | 3.95 | 3.70 | 4.30 | 4.40 | 2.90 | 2.33 | 3.89 | 1.73 |

* The latexes used are styrene butadiene, acrylic and styrene acrylic ester latex for SBR, A and SAE, respectively

** Includes the water present in the latex

Manufacturers of precast concrete slats normally use a properly compacted high-quality concrete with the ordinary Portland cement CEM I 42.5 R (nomenclature according to the Belgian standard NBN B12-001 1993, based on the European ENV 197-1). The concrete composition of one of those manufacturers was chosen for the reference concrete *Ref I*. Furthermore concrete with blastfurnace slag cement CEM III/A 42.5 (36-65% slag), which showed a superior performance in previous experiments (De Belie *et al*., 1996) was tested. For the concrete mixes *LS-G* and *LS-L*, half of the natural sand was substituted with limestone sand in order to neutralize the aggressive environment more quickly and reduce concrete attack afterwards. For *LS-L* also limestone aggregates were used instead of gravel. The Belgian Ministry of Agriculture advised in the past to use only rounded gravel aggregates for slatted floors in animal houses, which would reduce claw injuries compared to the sharp-edged limestone aggregates. Buist (1987) and Bayoux *et al*. (1990), however, claim that the limestone aggregates make an important buffer and that their use would therefore reduce acid attack. Furthermore the sharp edges would be rounded by the attack.

Table 2 shows that flow and slump of the concretes with limestone sand were very low, which aggravated compaction. This can be attributed to the shape of the sand and to the larger amount of particles smaller than 80 μm, compared to natural sand.

Four PCCs were tested with different polymer latexes: two styrene-butadienes *SBR1* and *SBR2*, an acrylic latex *A* and a styrene acrylic acid ester *SAE*. A polymer (d.m.)/cement proportion of about 0.1 was chosen. The polymers were added to the mixing water as latex. Normally polymers when added to concrete, exhibit water reducing qualities, because of the dispersing agents in the polymer latexes and the spherical shape of the polymer drops (Bijen 1991). The characteristics of hardened PCC are the result of the lower water-to-cement ratio and the formation of a three-dimensional polymer structure through the hardened cement paste. The nature of microstructural modification and void filling and bridging of cracks that occurs when polymer formulations are incorporated in cement systems, is such that polymers will substantially change, in a favorable manner, the pore structure. The porosity is decreased in the pore radius range of 240 nm or more, whereas it increases greatly in the smaller pore radius range of 140 nm or less. The net result of such changes would be to improve the resistance of PCC to liquids (Swamy 1995). However, the addition of polymers increases the concrete cost and would cause an increase in price of concrete slats with about 20-40%. Table 2 shows that slump and flow of *SBR1* and *SBR2* were quite high, which implies that the water content could be reduced further. In spite of the higher amount of water added to *A*, its workability was bad and slump and flow could not be measured. It is also clear that the presence of the polymer films inhibits water absorption. The *SAE*-concrete showed the smallest water absorption, but also for the *SBR1*- and *SBR2*-concretes the water absorption was remarkably lower than for the reference concrete. The *A*-concrete contained many air voids due to its bad workability, which explains for its higher water absorption.

*Experiment 2: sulfuric acid attack*

**Table 3. Composition and characteristics of the fresh and hardened (28 days old) concrete used in experiment 2**

| | *Concrete Mix* | | | | | | |
|---|---|---|---|---|---|---|---|
| | *Ref II* | *III/B* | *SBR* | *SAE* | *A* | *VPV* | *SF* |
| CEM I 42.5 HSR/LA (kg) | 350 | - | 350 | 350 | 350 | 350 | 350 |
| CEM III/B 42.5 HSR/LA (kg) | - | 350 | - | - | - | - | - |
| Silica fume (kg) | - | - | - | - | - | - | 30 |
| Sand (kg) | 840 | 840 | 843 | 839 | 843 | 819 | 840 |
| Gravel (kg) | 1120 | 1120 | 1124 | 1119 | 1124 | 1092 | 1120 |
| Latex* (kg) | - | - | 54.7 | 51.5 | 58.3 | 65.6 | - |
| Superplasticizer (kg) | 2.5 | 2.5 | - | - | - | - | 5 |
| Water** (kg) | 140.0 | 140 | 90.6 | 97.3 | 86.9 | 104 | 130 |
| W/C** (-) | 0.40 | 0.40 | 0.34 | 0.35 | 0.34 | 0.41 | 0.34 |
| Slump (mm/class) | 20/S1 | 15/S1 | 40/S1 | 40/S1 | 105/S3 | 75/S2 | 105/S3 |
| Flow (-/class) | 1.69/F2 | 1.26/F1 | 1.62/F2 | 1.47/F1 | 2.04/F3 | 1.71/F2 | 1.71/F2 |
| Fresh density (kg/m$^3$) | 2430 | 2400 | 2410 | 2370 | 2190 | 2370 | 2420 |
| Air content (%) | 3.0 | 3.8 | 4.4 | 4.0 | 9.7 | 5.2 | 3.6 |
| Compressive strength (N/mm$^2$) | 68.3 | 62.5 | 58.2 | 61.3 | 43.8 | 50.6 | 84.6 |
| Water absorption (%) | 2.58 | 2.28 | 1.56 | 1.84 | 2.28 | 2.71 | 1.78 |

* The latexes used are styrene butadiene, styrene acrylic ester, acrylic and vinylcopolymer latex for SBR, SAE, A and VPV, respectively

** Includes the water present in the latex

All concrete mixtures tested were made with 350 kg/m³ high sulfate resistant Portland cement (CEM I 42.5 HSR/LA), except for one mixture that was made with blast furnace slag cement CEM III/B 42.5 HSR/LA (66-80% slag). Four different polymer types were used: a styrene-acrylic ester polymer, an acrylic polymer, a styrene butadiene polymer and a vinylcopolymer. The polymer cement concrete mixtures were made with a polymer/cement ratio of 7.5% (polymer d.m./cement). The reference mixture without polymer was made with a W/C ratio of 0.4. For the mixtures with polymer, the water content was adjusted till a slump of class S1 (10 to 45 mm) was obtained (Belgian standard NBN B15-232 1982). Although the mixture with the acrylic polymer had a low W/C ratio (0.34), due to a high air entrainment of the fresh concrete, a slump of class S3 (100 to 150 mm) was obtained. Because the mixture with the vinylcopolymer was too sticky, it was not possible to work with an S1 class and more water had to be added to create a workable concrete mixture. The final W/C of the different mixtures was calculated taking into account the water content of the added latexes. All polymer modified concrete mixtures had a lower density than the reference mixture without polymer. This was mainly due to the higher air entrainment of the concrete with addition of polymer. The addition of the styrene butadiene polymer, the styrene-acrylic ester polymer and the vinylcopolymer caused an increase in air content of the fresh mixture by 1 to 2% compared to the reference mixture (*Ref II*). The addition of the acrylic polymer resulted in an increased air content of almost 10%. The silica fume mixture was made by adding 30 kg/m³ silica fume to the reference mixture. The low w/c ratio (0.34) of this mixture combined with the addition of a high amount of superplasticizer created very dense and still good workable concrete.

# 3 RESULTS

## 3.1 Experiment 1: lactic/acetic acid attack

*Average attack depth*

An analysis of variance was carried out with the average attack depth of the cylinder profiles as the dependent variable and the concrete composition (eight classes), the cylinder (three classes) and the profile (seven classes) as independent variables. Only interactions of second order were taken into account. The average attack depth of the profiles closest to the troweling surface was often smaller than for the other profiles. Some segregation could have taken place during compaction, which resulted in less aggregates in the upper concrete layer, coming loose by chemical attack and brushing. When these profiles were left aside, the concrete composition remained the only significant factor. Fig. 2 shows the accumulated average attack depth of the different concrete types versus the number of attack cycles.

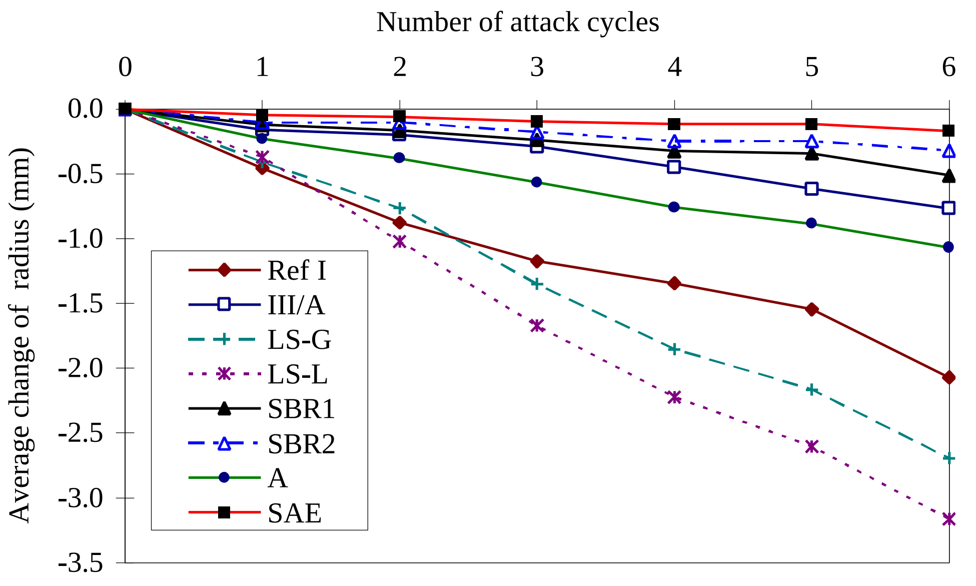


**Figure 2. Average attack depth of the different concrete types of experiment 1 (lactic/acetic acid) vs. number of attack cycles**

Already after the first attack cycle, the PCCs and the concrete with blastfurnace slag cement appeared to be significantly more resistant than the reference concrete and the concretes with limestone sand (Student-Newman-Keuls test with level of significance = 0.05). This difference increased after every cycle. Also the polymer cement concrete *A* appeared to perform significantly worse than the other PCCs. This could be expected from the difficulties encountered during compaction of *A* and the inferior physical characteristics. From cycle three onwards the average attack depth of the concretes with limestone sand surpassed the average attack depth of the reference concrete. Furthermore the concrete with limestone sand and limestone aggregates was more degraded than the one with limestone sand and gravel. The neutralizing effect of limestone sand and limestone aggregates was insufficient to limit the total attack. After six attack cycles the average attack depths for the different concrete compositions were all significantly different and varied between 0.17 mm for *SAE* and 3.16 mm for *LS-L*. The vulnerability to attack by lactic and acetic acid increased in the following order: PCC with the styrene acrylic acid ester *SAE*; PCC with the styrene butadiene latex *SBR2*; PCC with the styrene butadiene latex *SBR1*; concrete with blastfurnace slag cement III/A; PCC with the acrylic latex *A*; reference concrete with ordinary Portland cement *Ref I*; concrete with lime sand and gravel aggregates *LS-G*; concrete with lime sand and limestone aggregates *LS-L*. Modification of the concrete with styrene acrylic ester therefore seemed to be the best solution, even though investigation with the scanning electron microscope showed that the emulsified polymer had not completely formed a film. This would be due to the relatively high minimum film forming temperature of this polymer (32°C). Comparison with an on-farm trial (De Belie 1997b) revealed that the degradation of the reference concrete after two attack cycles of the TAP was similar to the attack of concrete slats in houses for fattening pigs after 9 months in front of the wetfeeders.

*Average Surface Roughness*

The most important factor influencing the $R_a$-value, was again the concrete composition. Fig. 3 shows the $R_a$-value of the different concrete types versus the number of attack cycles. Initially *SAE* showed a significantly larger surface roughness ($R_a$ = 0.10 mm) than the other concretes, except *LS-L* (Student-Newman-Keuls test with level of significance = 0.05). The subsequent increase in roughness was however extremely small ($R_a$ = 0.18 mm after six cycles). The increase in surface roughness of the two PCCs with styrene-butadiene latex, *SBR1* and *SBR2*, was also limited. The $R_a$-value of *A* surpassed the others till cycle four; then *LS-G* began to show the largest surface roughness. The surface of the *LS-L* concrete remained much smoother, due to the equal erosion of limestone aggregates and cement paste. After the six attack cycles the surface roughness increased as follows: PCC with the styrene acrylic acid ester *SAE*; PCC with the styrene-butadiene latexes *SBR2* or *SBR1*; concrete with limestone sand and limestone aggregates *LS-L*; concrete with blastfurnace slag cement *III/A*; PCC with the acrylic latex *A*; reference concrete with ordinary Portland cement *Ref I*; concrete with limestone sand and gravel aggregates *LS-G*.

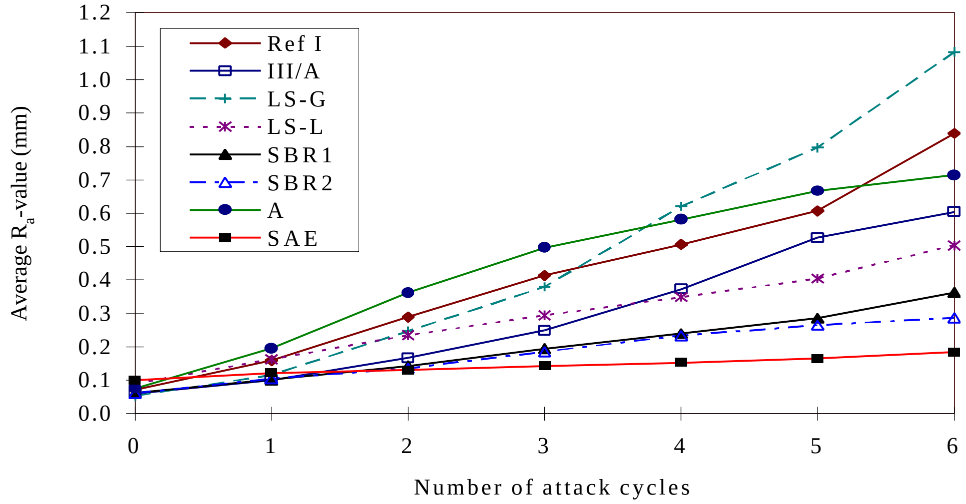


**Figure 3. Average surface roughness of the different concrete types of experiment 1 (lactic/acetic acid) vs. number of attack cycles**

### 3.2 Experiment 2: sulfuric acid attack

*Average change in radius*

In Fig. 4 the average change of the radius of the cylinders is shown versus the number of attack cycles for the different concrete mixtures. For every cycle two measurements were performed: one before and one after brushing the cylinders. A positive value represents an expansion of the cylinders compared to the initial size, while a negative value means that due to loss of material, the radius of the cylinder decreased compared to the initial dimensions. The alternating increase and decrease of the radius corresponds to alternating expansion of the concrete due to immersion and formation of reaction products and subsequent material loss due to brushing.

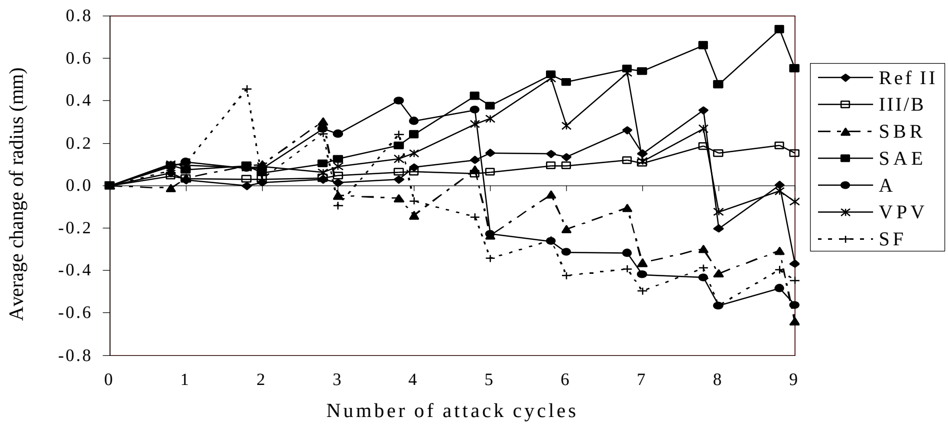


**Figure 4. Average change of radius of the different concrete types of experiment 2 (sulfuric acid) vs. number of attack cycles**

From the fifth cycle on the different mixtures could be classified in at least two groups (student-Newman-Keuls test with level of significance 0.05). One group included the reference mixture, the mixture with blast furnace slag cement and the mixtures with the styrene acrylic ester polymer and the vinylcopolymer. The other group, which appeared to be more vulnerable to degradation, included the mixtures with acrylic polymer, styrene butadiene polymer and silica fume. The reference mixture and the mixture with vinylcopolymer showed only after eight cycles a decrease of the radius relative to the initial radius. The mixtures with the blast furnace cement and the styrene acrylic ester polymer still showed a resulting expansion after nine cycles. For the mixture with the styrene acrylic ester polymer the increase of the radius was significantly larger (0.6) mm than for the mixture with blast furnace slag cement (0.2 mm). After nine cycles the reference mixture joined up with the second group, including also the mixture with silica fume, the mixture with the acrylic polymer and the mixture with the styrene-butadiene polymer.

Figure 5 shows the cumulative changes of the average radius of the different mixtures due to brushing of the cylinders. These values represent the sum of all differences between the radius measured after brushing and the radius measured before brushing during the same cycle. This figure, excluding the effect of expansion during immersion in the sulfuric acid solution, makes more clear that especially the concrete types with blastfurnace slag cement or styrene acrylic ester polymer are quite resistant to abrasion. The compositions with silica fume or styrene butadiene showed the most severe loss of material. Especially the results of the silica fume mixture were below expectations. In general silica fume addition induces an increased resistance of concrete to chemical aggression, due to a refinement of the pore structure, a reduced permeability, a reduction of the calcium hydroxide content and an increase of the content of calcium silicate hydrates. Many researchers have found positive results in the specific case of concrete subjected to sulfate attack by adding silica fume to the concrete mix. The results of compressive and water absorption tests (Table 3) indicate that also in our case the silica fume concrete was of high quality. Nevertheless there are some other investigations which show little or no positive effect of silica fume addition with respect to sulfate attack. Neville (1981) stated that in case of sulfate attack, the influence of silica fume is ambiguous. Talero (1996) found that ettringite formation associated with the reaction of puzzolans occurred much faster than with the ordinary hydration products. This resulted in an increased corrosion rate. Another reason might be that the pore refinement leads to a lack of expansion space for the voluminous reaction products gypsum or ettringite. Scherer & Fidjestöl (1995) use the same theory to explain why a greater water-cement ratio (>0.5) can result in a decrease in degradation by sulfates.

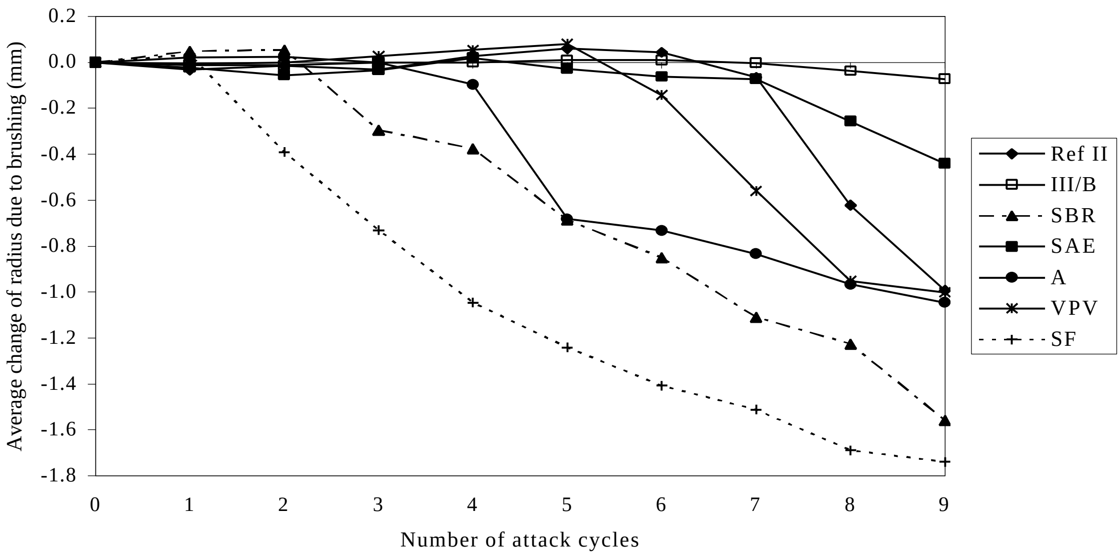


**Figure 5. Cumulative change of the average radius of the concrete types of experiment 2 (sulfuric acid) due to brushing**

The average $R_a$-value of the cylinders versus the number of cycles is shown in figure 6. The specimens with blast furnace slag cement showed a limited change in roughness. For all other mixtures, with exception of the mixture with the acrylic polymer, a gradual increase was measured during the subsequent cycles. The mixture with the acrylic polymer showed a peak after cycle 4 followed by a decrease of the roughness after cycle 5 and 6. This peak corresponds with a large expansion of the cement paste, followed by a significant amount of matrix material being detached. By this detachment the difference in level between the cement paste and the aggregates was again reduced.

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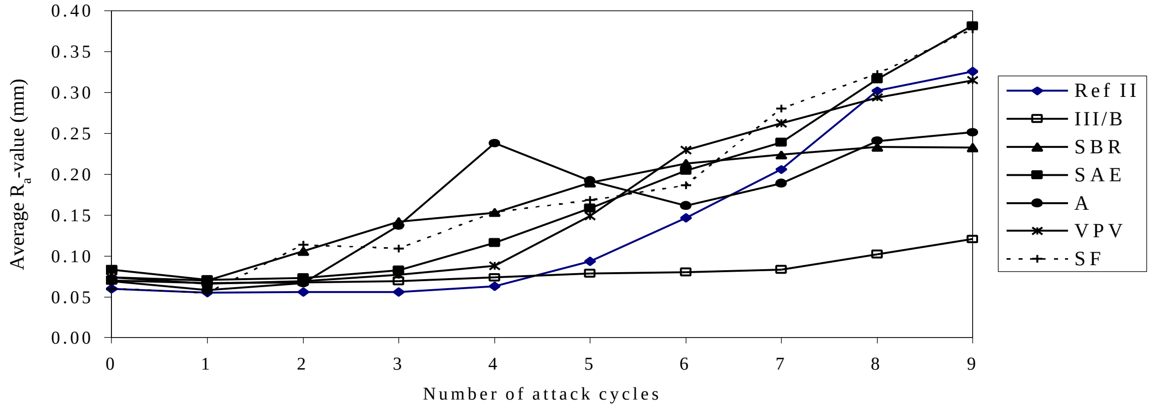


**Figure 6. Average surface roughness of the different concrete types of experiment 2 (sulfuric acid) vs. number of attack cycles**

### 3.3 Comparison of lactic/acetic and sulfuric acid attack

De degradation mechanism is completely different for the two liquids. The reaction between concrete and the organic acids involves an acid-base reaction with formation of soluble salts which are leached. Because of the continuing removal of free lime and the associated pH-drop the calcium silicate hydrates become unstable. The concrete surface is weakened and surface material is easily detached by brushing. On the contrary, the sulfuric acid attack is mainly described as the formation of expansive reaction products such as gypsum and ettringite. The latter is formed by reaction between calcium sulfate and the tricalcium aluminate of the cement and contains 32 water molecules in its structure. The increase in internal pressure results in development of cracks. Although some authors mention that also calcium lactates can cause an internal pressures (Kleinlogel 1960), no expansion was noticed during our experiments. In addition no cracks were found during the microscopic examinations on thin sections and with scanning electron microscopy (De Belie *et al*. 1997b). Figure 4 shows that the expansion linked to sulfuric acid attack was relatively large: up to 1.3 mm cumulative expansion over the 9 cycles for the silica fume concrete and about 0.7 mm for the reference concrete with Portland cement, even though high sulfate resistant cements were used in all compositions.

The organic acids used are much weaker than sulfuric acid, and the associated pH amounted to 2 compared to a pH of 1 for the sulfuric acid solution. Also the attack cycles involved a 6 day immersion for the organic acid attack and a 12 day immersion for the sulfuric acid attack. Nevertheless, the degradation caused by these organic acids was more severe. The reference concrete with Portland cement showed a decrease in radius of about 2 mm after 6 attack cycles of 6 days in the lactic/acetic acid solution (Fig. 2) and only 1 mm after 9 cycles of 12 days in the sulfuric acid solution (Fig. 5: cumulative decrease in radius without taking the expansion into account). Also the surface roughness at the end of the experiment was larger for the reference concrete subjected to lactic/acetic acid (about 0.8 mm compared to about 0.3 mm for the sulfuric acid solution). Apart from the different reaction mechanism, also the difference in acid concentration may be a reason for this effect (60 g lactic + acetic acid per liter simulation liquid compared to 5 g sulfuric acid). Bayoux *et al*. (1990) also mention that for weak acids such as lactic and acetic acid the acceptable pH limits are higher than for strong acids, because weak acids must be present in a higher concentration to cause the same pH. However, it must be noted that the concentrations used were realistic concentrations occurring on floors in animal houses and in sewer systems, respectively.

## 4 CONCLUSIONS

De resistance of different concrete compositions to aggressive solutions, firstly with lactic and acetic acid, and secondly with sulfuric acid, was tested with an apparatus for accelerated degradation tests (TAP). The aim was to simulate degradation in agricultural environments: on floors in animal houses and silage storage structures, and in manure storage facilities, respectively. It appeared that the degradation by the organic acids proceeded much faster, although the associated pH was higher. The degradation mechanism was also completely different, and involved mainly a leaching action for the lactic/acetic acid attack and the formation of expansive compounds for the sulfuric acid attack.

In both cases the use of blastfurnace slag cement instead of Portland cement was beneficial, even though high sulfate resistant Portland cement was used in the case of sulfuric acid attack. The effect of polymer modification depended highly on the type of polymer used. Addition of a styrene acrylic acid ester was the optimal solution in the case of lactic/acetic acid attack, whereas it resulted in a high expansion and increase in surface roughness in the case of sulfuric acid attack. Nevertheless, this expansive reaction was not associated with a loss of structural integrity and the detachment of material by abrasion was very limited. Modification with styrene butadiene polymer gave the second best results in the case of lactic/acetic acid attack, where it had rather a negative effect in the case of sulfuric acid attack. Acrylic polymer addition did help in neither case. Modification with vinylcopolymer was only tried in the case of sulfuric acid attack, but did not lead to a clear improvement, and silica fume addition caused a decrease in resistance.

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## 5 ACKNOWLEDGMENTS

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# Effect Of Water/Cement Ratio And Curing Temperature On Early-Age Shrinkage And Self-Induced Stresses Of High Performance Concrete

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**Summary: This paper presents the results of an experimental study dealing with the influence of water/cement (w/c) ratio and curing temperature on the autogenous deformations and self-induced stresses in early-age concrete. Two concrete mixes, w/c ratio 0.33 and 0.37, were cured isothermally at 10, 20, 30 and 40°C. The linear deformations of a free specimen and the self-induced stresses of a fully restrained one were measured in the first six days after casting. The lower w/c ratio mixture experienced both higher deformations and higher self-induced stresses. Higher temperatures, on the contrary, did not always lead to higher deformations in the observed period, but generally caused a faster shrinkage and a faster development of self-induced stresses. Numerical simulation of hydration at different temperatures allowed identifying in the different porosity of the pastes one of the causes of the unsystematic dependency of autogenous deformation on curing temperature.**

**Keywords: Water/cement ratio, Curing temperature, Autogenous shrinkage, Self-induced stresses.**

## 1 INTRODUCTION

In High Performance Concrete mixtures, a low w/c ratio leads to a significant self-desiccation during hydration (Persson & Fagerlund 1997). Addition of silica fume to the mix further increases this process (Jensen 1993). With the progress of hydration, the relative humidity in the pores of the hardening cement paste drops and the paste shrinks. Experimental correlations were found between relative humidity decrease and autogenous shrinkage (Jensen & Hansen 1996), but the actual process leading to autogenous deformation is not yet fully understood. The different mechanisms involved, namely capillary tension in the pore water, surface tension in the free-adsorbed water layer and disjoining pressure in the hindered-adsorbed water layer, were already pointed out by Powers (1965).

Though autogenous shrinkage increases for months (Baroghel-Bouny & Kheirbek 2000), the main part of the deformations and accompanying stresses take place in the first hours after setting, when the strain capacity is lowest (Kasai & Okamura 1968) and the elastic modulus has already reached substantial values. Since the elastic modulus of the aggregates is (in most cases) higher than that of the hardening paste, autogenous shrinkage results in tensile stresses in the cement paste (Golterman 1994). Furthermore, it results in bulk deformation of the concrete itself (Tazawa *et al.* 2000). Both these phenomena should be avoided as much as possible since they could induce micro- and macrocracking and impair the concrete quality (IPACS 2001).

The magnitude and development in time of autogenous shrinkage is fundamentally influenced by the w/c ratio. Baroghel-Bouny and Kheirbek (2000) observed increasing shrinkage in cement pastes with a reduction of the w/c ratio. Similar results were found for concrete (Persson 1998, Tazawa *et al.* 2000).

Measurements of autogenous deformation are normally performed at room temperature. A few authors have investigated the effect of different curing temperatures on autogenous shrinkage of concrete (Tazawa *et al.* 1995, Bjøntegaard 1999, Hedlund & Jonasson 2000, Lura *et al.* 2001). In their studies, a rather unsystematic influence of the curing temperature on the autogenous deformation was found.

Curing temperature and w/c ratio influence not only the deformations, but also the mechanical properties of young concrete, and thus both the self-induced stresses (which are higher the higher the stiffness) and the tensile strength. It is evident that when considering the risk of cracking of concrete at early age, the whole deformational and mechanical behavior, which depends on the development of the degree of hydration, should be taken into account.

In the present research, two concrete mixes, both realized with 475 kg/m$^3$ of blended cement (CEM I and CEM III) and 5% silica fume addition, differing only in the w/c ratio (0.33 and 0.37) were cured isothermally at 10, 20, 30 and 40°C. The linear free deformations in the first days after casting were measured with an Autogenous Deformation Testing Machine (ADTM). At the same time, on a fully restrained specimen (Thermal Stress Testing Machine, TSTM), the self-induced stresses were measured and the occurrence of cracking registered. Also the Young's modulus and the compressive strength were tested at different ages. The HYMOSTRUC model (Breugel 1997), calibrated with isothermal calorimetry, allowed the calculation of the degree of hydration of the mixtures and the parallel development of the microstructure in the cement paste. It must be pointed out that the degree of hydration concept (or an equivalent maturity concept) is not sufficient to describe the mechanisms that lead to autogenous deformation. A model able to simulate the development of the microstructure in the hardening pastes, especially the pore size distribution and the self-desiccation process, is needed. It will be shown in the following that HYMOSTRUC is able to simulate the effect of the curing temperature on the porosity of the cement paste, a first step towards the modeling of the whole self-desiccation process at different temperatures.

## 2 MATERIALS AND METHODS

### 2.1 Mix proportions

Two concrete mixtures with water/cement ratio 0.33 and 0.37, both with 5% silica fume addition, were studied (see Table 1). A blend of Portland cement (CEM I 52.5 R) and BFS cement (CEM III/B 42.5 LH HS) was used. The slag content of the BFS cement amounted to about 70%. The Blaine fineness of the two cements was 490 $m^2$/kg for the BFS cement and 530 $m^2$/kg for the Portland cement. Notice that the volumetric content of aggregate was 68% in mix A and 66% in mix B.

**Table 1. Mixture compositions of concrete with w/c 0.33 (A) and 0.37 (B)**

| *Mixture composition* | *A* | *B* |
|---|---|---|
| | *kg/m$^3$* | *kg/m$^3$* |
| CEM I 52.5 R (Portland cement) | 238.0 | 238.0 |
| CEM III/B 42.5 LH HS (BFS cement) | 237.0 | 237.0 |
| Water (including water in admixtures) | 125.4 | 175.8 |
| Crushed aggregate (4-16 mm) | 973.5 | 944.2 |
| Sand 0-4 mm | 796.5 | 772.5 |
| Lignosulphonate | 1.0 | 0.9 |
| Naphtalene sulphonate | 9.5 | 7.6 |
| Silica fume slurry (solid 50%) | 50.0 | 50.0 |

### 2.2 Cube compressive strength and modulus of elasticity

For the two mixtures the compressive strength and the modulus of elasticity were tested at different curing temperatures. The compressive strength was measured on concrete cubes, 150x150x150 $mm^3$; the modulus of elasticity in compression was tested on prisms, 100x100x400 $mm^3$. Cubes and prisms were cast in temperature-controlled steel moulds. The specimens tested at later ages were removed from the moulds after six days, sealed with plastic and aluminum foils and stored at constant temperature until the moment of testing.

### 2.3 Free deformations

Measurements of the free deformations were performed with an ADTM, shown in Fig. 1. The concrete was cast in a prismatic mould, 150x150x1000 $mm^3$, made with thin steel plates provided with an external insulating material. A foil was folded into the mould to prevent leakage of water from the fresh concrete and drying of the hardening concrete. A felt placed beneath the foil reduced the friction between the mould and the specimen. After casting, the top surface of the concrete was covered with a tight cover in order to avoid moisture loss to the environment. A wooden frame supported the mould during casting, vibrating and setting of the concrete. The mould could be cooled or heated by a system of tubes located between the plates and the insulating material. Temperature differences inside the concrete were kept as low as 1.5 K.

Length changes of the hardening concrete were measured between two steel bars embedded in the concrete. The bars passed through the mould and at their ends linear variable differential transducers (LVDTs), attached to the outer ends of quartz rods, registered the deformations. The autogenous deformation was measured until six days after casting.

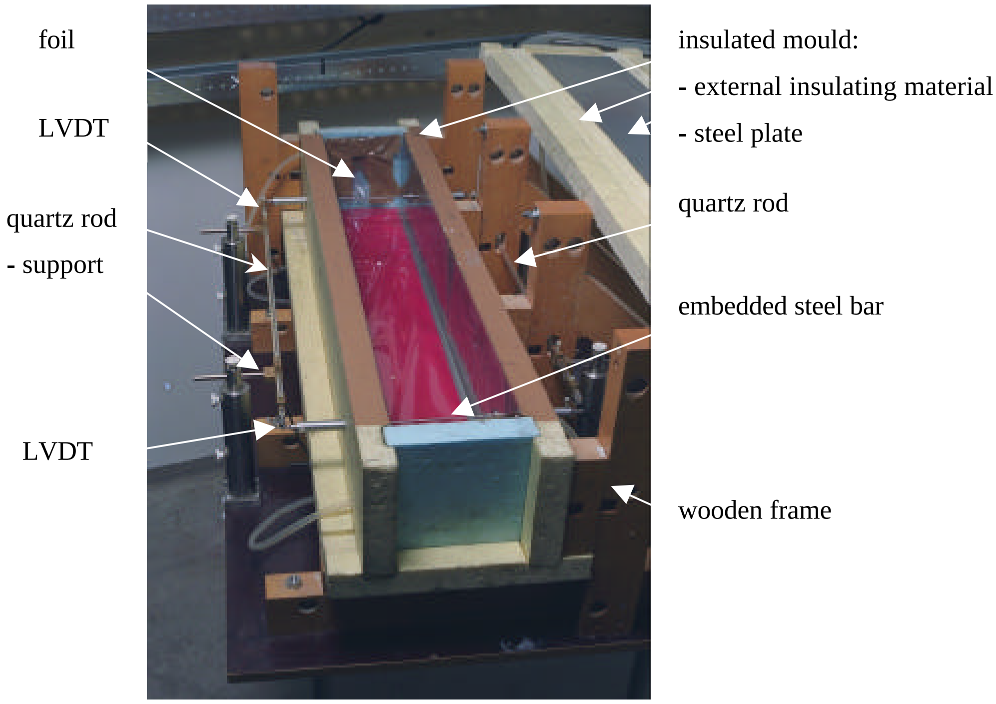



**Figure 1. Experimental setup for the measurement of the free deformations of hardening concrete (Autogenous Deformation Testing Machine).**

### 2.4 Self-induced stresses

The self-induced stresses were measured with a TSTM. It consisted of a horizontal steel frame (Fig. 2) where hardening concrete could be loaded in compression and in tension under various hardening conditions. With the help of a temperature controlled mould any curing temperature could be imposed. The specimen had a prismatic shape with dovetailed heads, where it was held by rigid steel claws. One of the claws was fixed to the frame; the other lied on roller bearings and could be moved with a hydraulic actuator. The claws could be prestressed to avoid slip of the specimen if the load changed from compression to tension. Loads were measured with a load cell. In this research the stress generated under total restraint was measured.

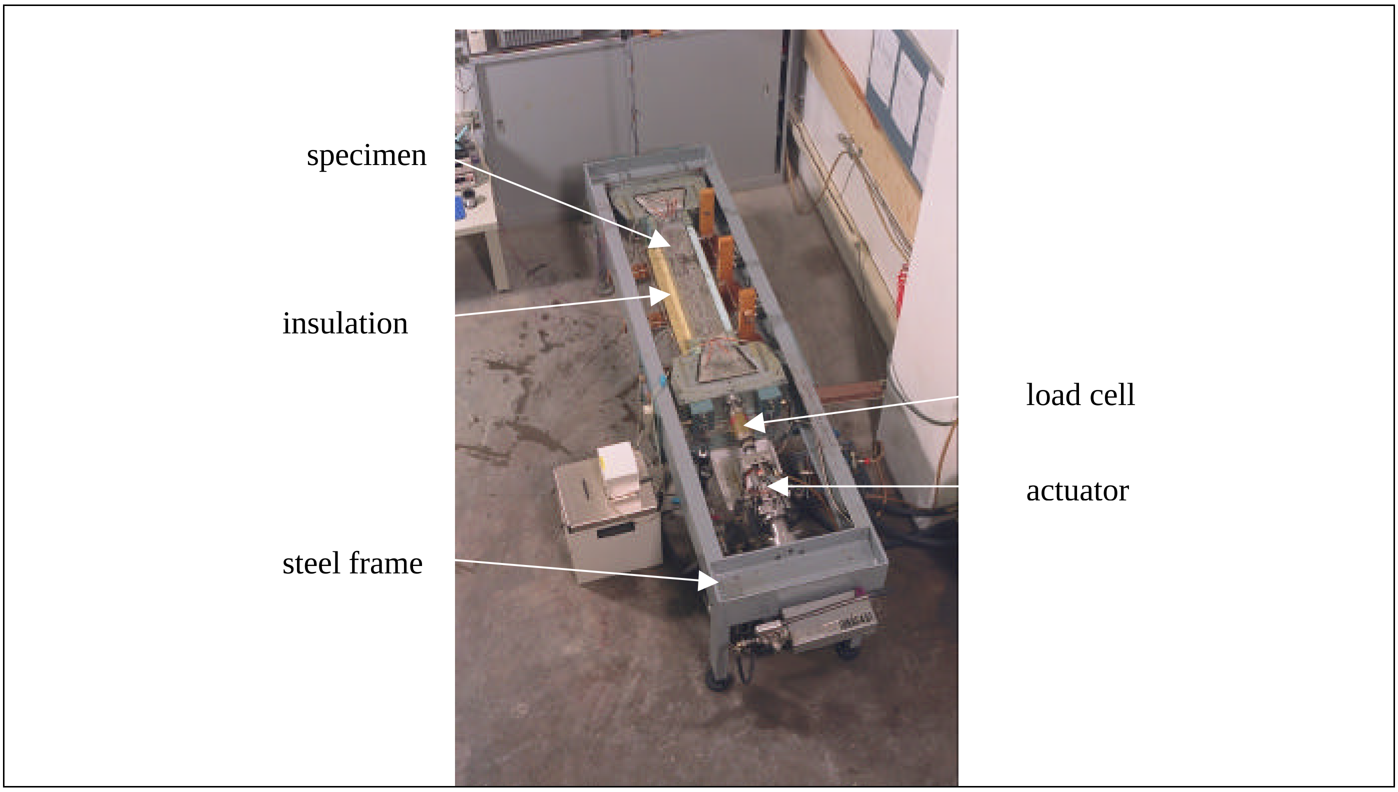



**Figure 2. Experimental setup for the determination of stress development in hardening concrete (Temperature Stress Testing Machine).**

# 3 RESULTS AND DISCUSSION

## 3.1 Cube compressive strength and modulus of elasticity

Results of compressive strength and elastic modulus are reported in Tables 2 and 3. The results represent the average value of three specimens.

**Table 2. Cube compressive strength of concrete with w/c ratio 0.33 and 0.37, cured at different temperatures**

| | | *Mean cube compressive strength [MPa]* | | | | | |
|---|---|---|---|---|---|---|---|
| *Mixture* | *Temperature* | *1day* | *2 days* | *3 days* | *7 days* | *14 days* | *28 days* |
| A | 20°C | 39.5 | 59.8 | --- | --- | --- | 105.0 |
| A | 30°C | 51.9 | 68.8 | 80.4 | 95.0 | --- | 103.5 |
| A | 40°C | 58.0 | 69.6 | --- | 95.7 | --- | 97.5 |
| B | 10°C | --- | 31.1 [1)] | 38.1 [2)] | 57.4 | 72.2 | 82.8 |
| B | 20°C | 42 | 52 | 58 | 70 | --- | 82 |
| B | 30°C | 40.7 | 54.5 | 63.9 | --- | 84.8 | --- |
| B | 40°C | 44.9 | 65.1 | 71.4 | 78.8 [3)] | 79.2 | --- |

[1)] Tested at 1.5 days [2)] Tested at 2.5 days [3)] Tested at 8 days

**Table 3. Elastic modulus of concrete with w/c ratio 0.33 and 0.37, cured at different temperatures**

| | | *Young's modulus [GPa]* | | | | | |
|---|---|---|---|---|---|---|---|
| *Mixture* | *Temperature* | *1day* | *2 days* | *3 days* | *7 days* | *14 days* | *28 days* |
| A | 20°C | 27.8 | 32.6 | --- | 37.0 | --- | 39.8 |
| B | 10°C | --- | 24.9 [1)] | 26.5 [2)] | 32.7 | --- | 37.3 |
| B | 30°C | 27.8 | 32.2 | 32.2 | --- | 39.5 | --- |
| B | 40°C | 30.7 | 34.5 | 34.2 | --- | 38.0 | --- |

[1)] Tested at 1.5 days [2)] Tested at 2.5 days

Mixture A, with a lower w/c ratio, shows higher strengths at 28 days and faster strength increase than mix B at all temperatures. Comparison of the strength results in mix A at different curing temperatures shows that, although higher temperatures promote the initial strength development, the value at a later stage (28 days), seems to be penalized, as already observed (Rakel 1965). The same results were found for mix B, where the strength at 14 days is higher at 30ºC than at 40ºC. For the specimen cured at 10ºC progress of hydration was slower, but the strength at 28 days was already slightly higher than for 20ºC curing.

The results of the elastic modulus show a slightly faster development in mix A, whereas the final value seems to be similar for the two mixes. It is noticed that the modulus of elasticity develops relatively faster than the compressive and the tensile strength (Rostasy *et al.* 1993). This fact potentially increases the cracking risk, since the concrete might develop stresses (due to restrained deformations) while the strength is still very low.

## 3.2 Free deformations and self induced stresses

### 3.2.1 Effect of w/c ratio

In Fig. 3 the autogenous deformation and the self-induced stress of mixtures A and B are plotted. The autogenous deformation was zeroed at the time when stress was first recorded in the TSTM. Thus, only the stress-inducing deformation is shown. It can be seen that under the same curing temperature (20°C), concrete with a lower w/c ratio (mixture A) underwent more autogenous deformation and developed self-induced stress slightly faster.

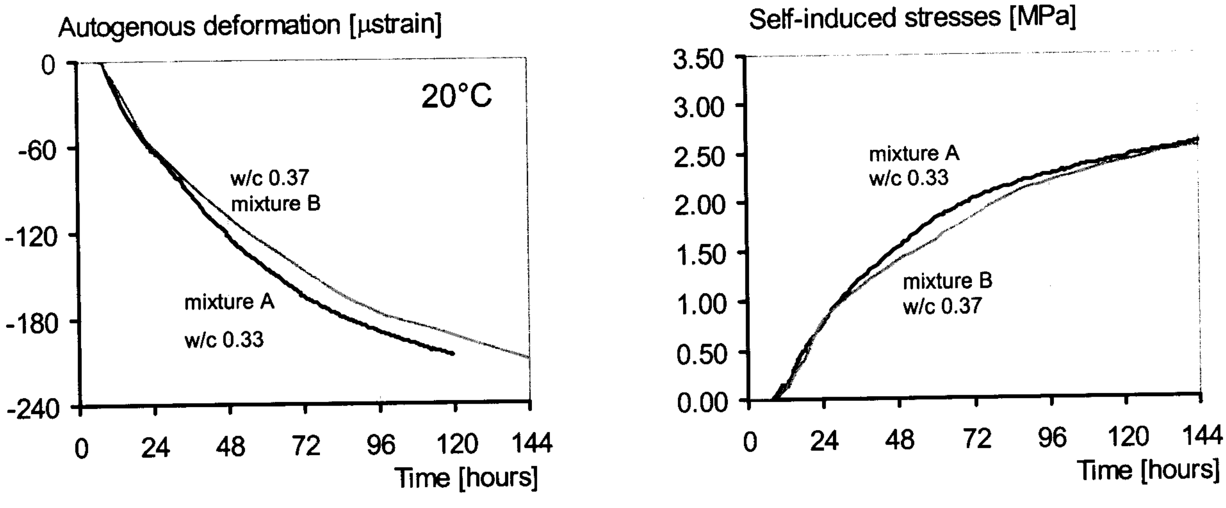
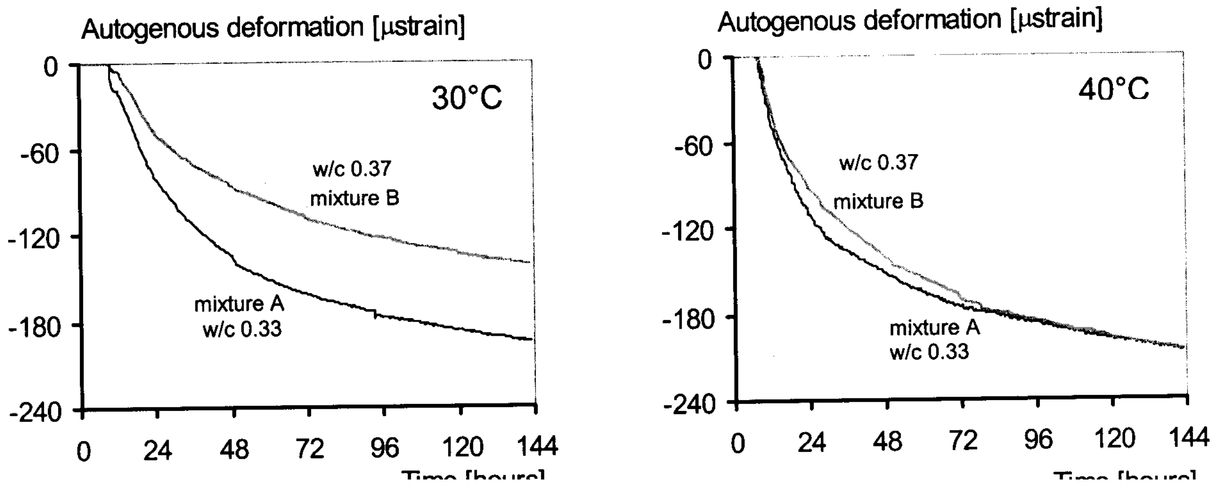



**Figure 3. Autogenous deformation (left) and stress development (right) of concrete mixtures with different w/c ratios cured at 20°C (mixture A: w/c 0.33, mixture B: w/c 0.37).**

In Fig. 3 (left) and in Fig. 4 the autogenous deformations of the two mixes at curing temperatures 20, 30 and 40°C are shown. It can be noticed that a lower w/c ratio leads to higher autogenous shrinkage. This effect is found at all temperatures, though it is most evident in the case of curing at 30°C.




**Time (hours)** **Time (hours)**

**Figure 4. Autogenous deformation in mixtures A and B at 30°C (left) and 40°C (right).**

The autogenous shrinkage of both mixtures reached values between 150 and 200 µstrain after 6 days. This value is in agreement with literature data on similar concrete mixtures; see for example Igarashi *et al.* (2000). For all curing temperatures the measured shrinkage was only slightly higher for the concrete with the lower w/c ratio (mixture A). One would expect a higher difference between the autogenous shrinkage of the two concretes, due to the difference in w/c ratio (0.33 vs. 0.37). A possible reason could be the scatter in the measurements. Differences of about 8% were found in repeated experiments. Another factor reduces the differences between the autogenous deformations of the two mixtures, though in a minor extent, is the higher aggregate content, which is 68% by volume in mixture A and 66% in mixture B. In fact, the shrinkage of concrete is inversely proportional to the aggregate content, as already found by Pickett (1956) for drying shrinkage and recently by Tazawa *et al.* (2000) for autogenous shrinkage. Finally, a factor that influences the autogenous shrinkage of cement pastes very much is the silica fume content. In experiments performed on cement paste, Jensen (1993) observed that addition of 6% silica fume to pastes of different w/c ratio led to two times higher shrinkage (about 1000 µstrain vs. 500 µstrain, for w/c ratio 0.35).

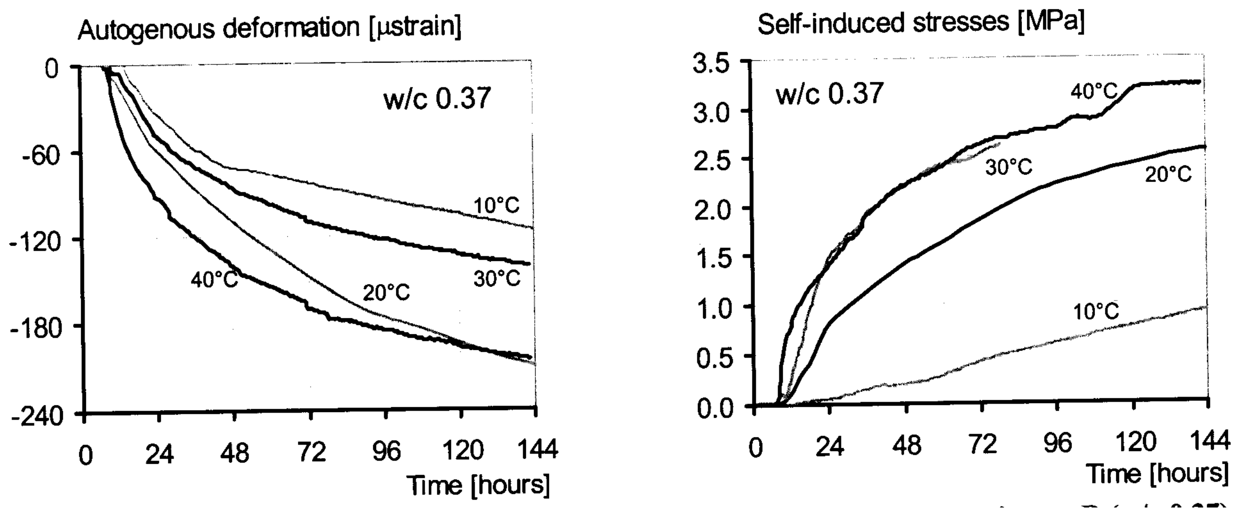



**Figure 5. Autogenous deformation (left) and self-induced stresses (right) in mixture B (w/c 0.37) at different temperatures. Note: the 20ºC curve refers to other batch (see text).**

The development of the self-induced stresses in time (Fig. 3, right) is also very similar in the two mixes. The lower w/c ratio leads to slightly higher tensile stresses in the first three days. Since the Young's modulus and the shrinkage of the two concretes did not differ substantially, this result was expected. The measured self-induced stresses are in agreement with results by Igarashi *et al.* (2000).

3.2.2 Effect of different temperatures on free deformation and self-induced stresses

The effect of the curing temperature on the free deformation and the self-induced stress can be observed in Fig. 5. Results of self-induced stress (Fig. 5, right) show that the higher the curing temperature the faster the stress developed. The free deformation did not show such a clear trend (Fig. 5, left). In fact, the shrinkage of the specimen cured at 20°C developed faster than in the case of 30°C curing. It must be pointed out, however, that the deformations at 20°C had been measured in a previous research (Breugel & Vries 1999); the materials used might have been slightly different (different cement batches, for example). Differences in the measured shrinkage could also be due to scatter in the results (tests at 10, 30 and 40°C were repeated, showing a difference of about 8%).

Higher temperatures, causing faster development of shrinkage and self-induced stresses (Fig. 5, right), might increase the cracking risk. In fact, the specimens cured at 30ºC cracked 80 hours after casting. This fact is also confirmed by results on other concrete mixes (Lura *et al.* 2001). Jensen and Hansen (2001) observed that the rate of the autogenous deformation is at least as important as its absolute value in determining the cracking risk. In fact, a slower development of self-induced stresses allows the relaxation process to reduce the stresses in the specimen. Reducing the rate of the autogenous deformations, thus, might contribute to reduce the cracking risk.

In order to give further insight in the influence of the curing temperature on the autogenous deformation, the hydration of mixture B was simulated with the program HYMOSTRUC (Breugel 1997). The program is able to simulate the development of the hydration process in a concrete mix knowing the mineral composition and the particles size distribution of the cement and the w/c ratio. In order to obtain a more accurate simulation, it is possible to calibrate the results on experimental data. In this research, data obtained with an isothermal calorimeter on cement paste were used.

A cube of hydrating cement paste (100x100x100 $\mu m^3$) was simulated. The cement particles were modeled as digitized spheres randomly distributed in a three-dimensional body and the hydrating cement grains were simulated as growing spheres. The development of the porosity and of the connectivity of the solid phase during hydration was obtained. The influence of the curing temperature on the morphology of the hydration products was simulated considering a temperature-dependent expansion factor ? of the hydration products (Breugel 1997). This variable expansion factor has been derived from experimental observations of capillary porosity at different temperatures (Bentur *et al.* 1979).

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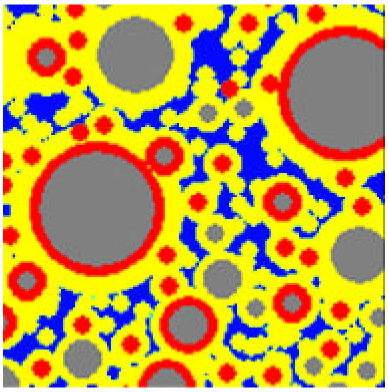

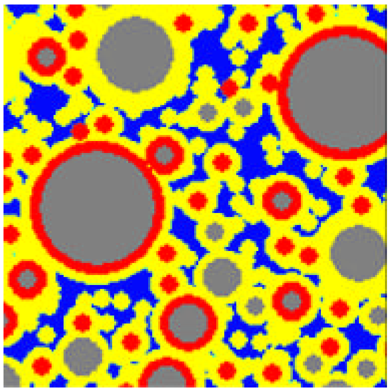

**Figure 6. Sections of simulated 3D microstructures of cement paste (mix B, w/c 0.37) cured at 20ºC (left) and 40ºC (right).**

Figure 6 shows a section of the 3D microstructure hydrated at 20ºC (left) and at 40ºC (right). Unhydrated cement is grey, inner product is red, outer product is yellow and capillary voids are blue. Even if the two pictures were taken at the same degree of hydration (a=0.31), it is noticed that the microstructure cured at 40ºC shows a thinner outer product, especially around the biggest cement particles. This results in fewer interparticle contacts and in higher porosity in the paste cured at higher temperature. 3D porosity calculations on the whole hydrating microstructure confirm this fact: the porosity is 11.7% for 20ºC and 13.1% for 40ºC curing.

This different porosity has a double effect on the autogenous deformations. On the one hand it accelerates the self-desiccation process (for the same amount of water reacted, a greater empty porosity is formed). On the other hand, the increased porosity will result mostly in bigger pores and the internal stresses due to capillary depression in the pore water will be lower. In fact, as resulting from the Laplace law (see for example Hua *et al.* 1995) the capillary depression is inversely proportional to the radius of the pore. The effect of the curing temperature on the pore size distribution and on the shape of the capillary pores will be object of further investigation.

The net effect of these phenomena is difficult to quantify. Nevertheless, it is important to point out that the effect of the curing temperature on morphology of the paste, internal stresses and deformability cannot be described only with a simple degree of hydration (or maturity) concept.

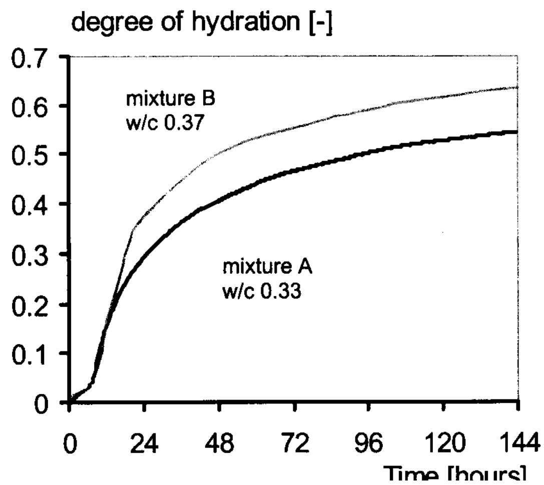


**Figure 7. Degree of hydration of mixtures A and B vs. time at 20ºC deduced from isothermal calorimeter data.**

3.2.3 Self-induced stresses as a function of time and of degree of hydration

The degree of hydration vs. time (Fig. 7) was calculated from isothermal calorimeter data for mix A and mix B cured at 20ºC. Figure 7 shows that in mix A, with a lower w/c ratio, the degree of hydration is about 15% lower than in mix B. The degree of hydration of the two concretes is still increasing after 144 hours; this fact is confirmed by increasing strength and stiffness at later ages (see Tables 2 and 3).

Once the development of the degree of hydration in time was obtained, the autogenous deformations and self-induced stresses could be related to it. In Fig. 8 the autogenous deformations (left) and the self-induced stresses (right) are shown for the two mixes cured at 20°C as a function of the degree of hydration. The autogenous deformation of mix A developed at a lower degree of hydration. This was expected, since a lower w/c ratio accelerates the occurrence of self-desiccation. Also the self-induced stresses (Fig. 8, right) developed at a lower degree of hydration, due to the higher restrained deformation and higher elastic modulus (see Table 3). In order to have a clearer picture of the build-up of the self-induced stresses, information about creep and relaxation of the mixes (an their dependence on the degree of hydration) would be needed (Lokhorst 1998).

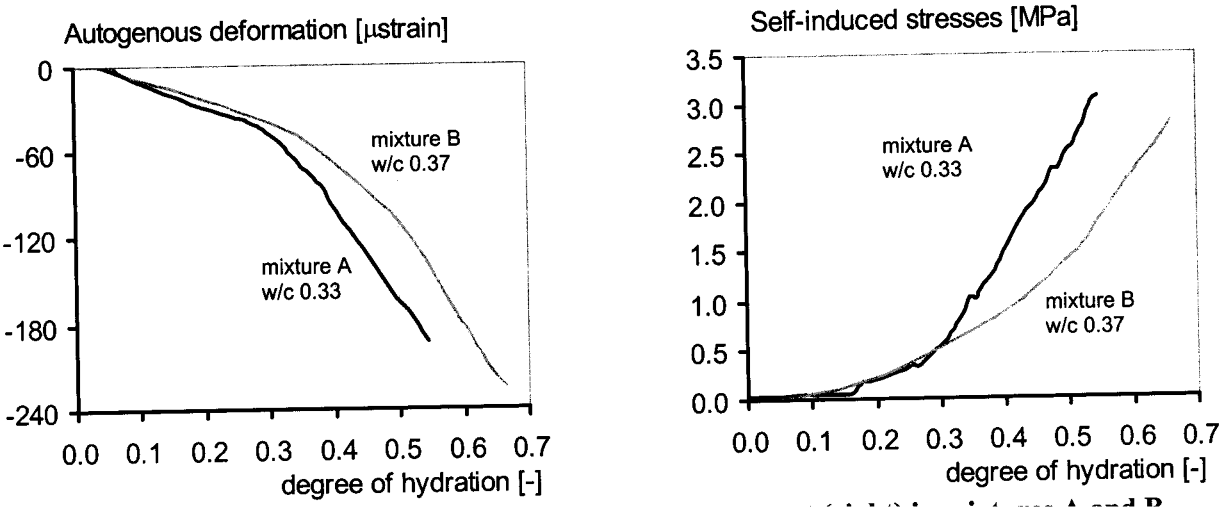


**Figure 8. Autogenous deformation (left) and stress development (right) in mixtures A and B, cured at 20°C, as a function of the degree of hydration.**

## 4 CONCLUSIONS

In this paper, results of an experimental study on autogenous shrinkage, self-induced stresses and mechanical properties of two concrete mixtures (w/c ratio 0.33 and 0.37) were presented. A summary of the findings of this study is provided below:

- The mixture with lower w/c ratio (mixture A) showed a higher strength and a faster development of the elastic modulus than mixture B at all the curing temperatures. For both concretes it was found, in agreement with findings by other authors, that the higher the curing temperature, the lower the strength at later ages.
- The mixture with lower w/c ratio (mixture A) showed substantially higher autogenous shrinkage and self-induced stresses. Since the development of the degree of hydration for mixture A was slower than for mixture B, the deformations and the stresses developed at a lower degree of hydration. In fact, due to the lower w/c ratio, mixture A undergoes self-desiccation (and consequent autogenous shrinkage) at a lower degree of hydration.
- The influence of curing temperature on the autogenous deformation was not systematic. However, the higher the curing temperature, the faster the self-induced stresses developed.
- Cracking was observed for specimens cured at higher temperatures when tested in the TSTM frame. The rate of the autogenous deformation is fundamental for the cracking risk, since a slower development of self-induced stresses allows the relaxation process to reduce the stresses in the concrete. High curing temperature, increasing the speed of the deformations, might therefore be detrimental for the risk of cracking.

The influence of the curing temperature on the microstructure of the cement paste was simulated with the HYMOSTRUC model. It was found that at the same degree of hydration, a higher porosity and coarser pores were found for the higher curing temperatures. The total porosity and the pore size distribution influence both the internal stresses due to self-desiccation and the deformability of the paste. Thus, a different morphology of pastes cured at different temperatures might explain the unsystematic dependency of autogenous shrinkage on curing temperature, which cannot be described with the degree of hydration concept alone.

## 5 ACKNOWLEDGMENTS

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# Volume Changes Of Lightweight Aggregate Concrete At Early Ages: The Effect Of Particle Size Distribution Of The Lightweight Aggregates In Experiments And Numerical Simulations

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**Summary: This paper deals with early-age volume changes and moisture transport in Lightweight Aggregate Concrete (LWAC). Three LWAC mixtures with water/cement (w/c) ratio 0.37 were tested, differing in the grading of the lightweight aggregates (LWA), while the quantity of water absorbed in the LWA was kept constant. Autogenous deformations, self-induced stresses in 100% restrained conditions and compressive strength were measured. It was found that a concrete with fine particles swells more than one with coarser particles. This finding was confirmed by modeling the concrete mixtures with analytical formulas.**

**In order to proceed towards the modeling of water transport from the LWA to the cement paste, absorption-desorption properties of the LWA and their volume stability in consequence of the moisture loss towards the matrix have also been studied. It was found that not all the water entrained in the LWA is available for the internal curing and that the LWA show moderate shrinkage in consequence of moisture loss.**

**Keywords: Lightweight Aggregates, Autogenous deformation, Self-induced stresses, Water transport, Internal curing**

## 1 INTRODUCTION

High-Performance Concrete (HPC) is generally made with a low w/c ratio, which makes it susceptible to self-desiccation (Persson & Fagerlund 1997). The addition of silica fume to the mix, to increase the strength and reduce the porosity, further increases the relative humidity (RH) drop in the concrete (Jensen & Hansen 1996). It has been shown (Jensen 1993) that autogenous shrinkage is directly related to the RH decrease. Autogenous shrinkage results in tensile stresses in the cement paste, due to the internal restraint of the aggregates (Golterman 1994), and in bulk deformation of the concrete itself (Tazawa *et al.* 2000). Both these phenomena should be avoided as much as possible since they could induce micro- or macrocracking and impair the concrete quality. A very efficient way to reduce or even eliminate autogenous shrinkage is the use of saturated LWA in the concrete (Breugel & Vries 1999).

In low water/binder ratio LWAC, water transport between the porous saturated aggregates and the matrix influences the hardening process (Weber & Reinhardt 1997). Due to self-desiccation of the matrix during hydration, a RH-gradient takes places within the concrete and a moisture flow from aggregate particles to the matrix may occur (Takada *et al.* 1998). As a consequence, the relative humidity in the matrix remains higher than in Normal Weight Concrete (NWC).

Of primary importance in this internal-curing process is the spatial distribution of the "water reservoirs" in the mixture. The distance of the saturated LWA from the point where the RH-drop takes place determines the efficiency of the internal curing. The maximum transport distance of water, as a consequence of the depercolation of the pores in a low w/c ratio paste, was estimated as hundreds of micrometers (Bentz & Snyder 1999). If the water-reservoirs are well distributed within the matrix, shorter distances have to be covered and the efficiency of the internal-curing process is increased. These considerations lead to the choice of small LWA.

In the present research, three LWAC mixtures with w/c ratio 0.37 were tested, differing for the grading of the LWA. The quantity of water absorbed in the LWA was kept constant. Autogenous deformation, self-induced stresses in 100% restrained conditions and compressive strength were measured. Analytical analysis of the effect of different particle size distributions of the LWA on the internal curing process was performed using the formulas by Lu & Torquato (1992). The percentage of paste within the assumed sphere of influence of the moisture transport from the LWA particles was estimated.

Two other important factors, in order to investigate the early-age swelling in LWAC, are the absorption properties of the LWA and the volume stability of the particles in consequence of the moisture loss towards the matrix. Tests on the aggregate particles have been carried out in order to quantify the water absorption, the desorption isotherm and the deformational response to changes in the water content. Finally, the amount of entrained water needed to totally avoid self-desiccation (and thus autogenous shrinkage) and the origin of the autogenous swelling in LWAC are also addressed in this contribution.

## 2 MATERIALS AND METHODS

### 2.1 Mix proportions

Three LWAC mixtures with water/cement ratio 0.37 and addition of 5% silica fume (in slurry form, 50% solid) were studied (Table 1). A blend of Portland cement (CEM I 52.5 R) and blast furnace slag (BFS) cement (CEM III/B 42.5 LH HS) was used. The slag content of the BFS cement amounted to about 70%. The Blaine fineness of the two cements was 490 $m^2$/kg for the BFS cement and 530 $m^2$/kg for the Portland cement. The LWA in the three mixes were Liapor F8 in two of the mixes and Liapor sand in the third mix. The grain density of the LWA is about 1500 kg/$m^3$ for Liapor F8 and 1400 kg/$m^3$ for Liapor sand. In all the mixes normal siliceous sand (1-4 mm) was used. Two types of plasticizers were added, i.e. lignosulphonate and naphthalene sulphonate.

**Table 1. Mixture compositions of LWAC with w/c ratio 0.37**

| Mixture composition | | Type of lightweight aggregate | | |
|---|---|---|---|---|
| | | Liapor F8 4-8mm | Liapor F8 8-16mm | Liapor sand 0-4mm |
| CEM III/B 42,5 HL HS (CEMIJ) | kg/$m^3$ | 237.00 | 237.00 | 237.00 |
| CEM I 52,5 R (ENCI) | kg/$m^3$ | 238.00 | 238.00 | 238.00 |
| Water (including water in admixtures) | kg/$m^3$ | 175.75 | 175.75 | 175.75 |
| Lightweight aggregate | kg/$m^3$ | 545.90 | 586.53 | 385.00 |
| Sand 0-4 mm | kg/$m^3$ | 772.51 | 772.51 | 772.51 |
| Lignosulphonate | kg/$m^3$ | 0.95 | 0.95 | 0.95 |
| Naphtalene sulphonate | kg/$m^3$ | 7.13 | 7.13 | 7.13 |
| Silica fume (slurry, 50% solid) | kg/$m^3$ | 50.00 | 50.00 | 50.00 |

The quantity of water absorbed in the LWA was about 15% by weight. This amount corresponded to the quantity absorbed after one day under water by the fraction of aggregate with the lowest absorption capacity (Liapor F8 8-16 mm). This quantity does not correspond to full saturation, as will be shown in the following. Being the amount of the LWA different in the three mixes, the quantity of absorbed water is also different. The difference is quite relevant in the case of Liapor sand (58 kg/$m^3$) while the other two mixes are almost equivalent in this respect (82 and 88 kg/$m^3$).

### 2.2 Cube compressive strength

The compressive strength was measured at 3 and 28 days after casting, on sealed cubes, 150x150x150 $mm^3$. The specimens were cast in temperature-controlled steel moulds. The cubes tested at 28 days were removed from the moulds after two weeks, sealed with plastic and aluminum foils and stored at constant temperature (20±1ºC) until the moment of testing.

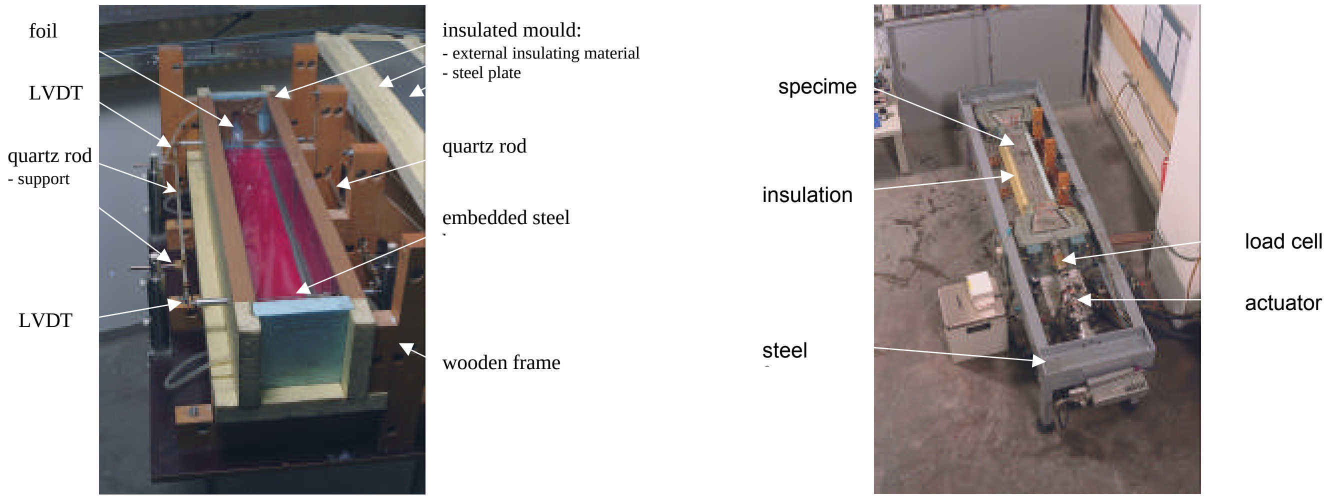



**Figure 1. Left, experimental setup for the measurement of the free deformations of hardening concrete (ADTM). Right, experimental setup for the determination of stress development in hardening concrete (TSTM).**

### 2.3 Free deformations

Measurements of the free deformations were performed with an Autogenous Deformation Testing Machine (ADTM), shown in Figure 1, left. The same device had been already used to measure autogenous deformation on NWC (Lura *et al.* 2001) and on LWAC mixes (Takada *et al.* 1998). The concrete was cast in a prismatic mould, 150x150x1000 $mm^3$, made with thin steel plates provided with an external insulating material. After casting, the top surface of the concrete was covered with a tight cover in order to avoid moisture loss to the environment. The mould could be cooled or heated by a system of tubes located between the plates and the insulating material. Temperature differences inside the concrete were kept as low as 1.5 K. A felt was used to minimize the friction between the sample and the mould. Length changes of the hardening concrete were measured with two external quartz rods provided with linear variable differential transducers (LVDTs) at both ends. The quartz rods were connected to steel bars cast in the concrete and measured displacements over a length of 750 mm. The rods could be fixed to the cast-in bars when the concrete had reached sufficient strength to support them (see Fig. 1, left). The autogenous deformation up to two weeks after casting was recorded.

### 2.4 Self-induced stresses

The self-induced stresses were measured with a Thermal Stress Testing Machine (TSTM). This device consisted of a steel frame (Fig. 1, right) supporting a mould in which the concrete was cast. The specimens had a prismatic shape with dovetailed heads. Two rigid steel claws held the specimen and were able to exert a tensile or a compressive force. One claw was fixed to the steel frame and one lied on roller bearings and could be moved with a hydraulic actuator. A temperature-controlled mould allowed realizing isothermal conditions. The imposed strain on the concrete in the TSTM was assumed equal to the free deformations measured in the ADTM, except for the sign. The control of the device was fully automatic. In this research, stresses obtained under total restraint were measured.

## 3 EXPERIMENTS ON LIGHTWEIGHT AGGREGATE CONCRETE

### 3.1 Cube compressive strength

Results of compressive strength are shown in Figure 2. The results represent the average value of three specimens. The strength at 28 days of the mixtures with Liapor F8 was considerably higher than the strength of the mixture with Liapor sand (80 MPa versus 65 MPa). It is noticed that the three mix compositions were meant to be equivalent in every respect except for the particle size of LWA. This is true only for the two mixes with Liapor F8 4-8 mm and 8-16 mm, which show rather similar strength both at 3 and 28 days. For the same mix realized with normal weight aggregates (Lura *et al.* 2001), also shown in Fig. 2, the compressive strength was 58 MPa at 3 days and 82 MPa at 28 days. Thus, high performance LWAC reached 28 days-strength similar to NWC with the same matrix.

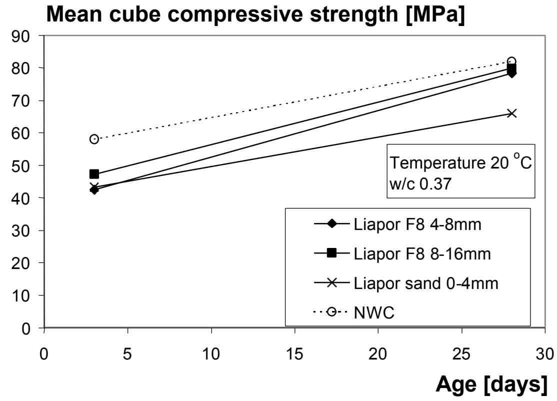



**Figure 2. Cube compressive strength of mixtures with LWA of different particle size.**

Since the mechanical properties of the LWA are generally inferior to the ones of NWA (Nilsen *et al.* 1995), this fact might be due to an improved quality of the interfacial transition zone and of the bond between LWA and matrix. It has been observed (Zhang & Gjørv 1991, Wasserman & Bentur 1996), that cement paste penetrates in the outer porous layer of the LWA particles, strengthening the bond.

Continuous hydration of the mixture at later ages, promoted by the extra water stored in the LWA, might also contribute to the strength increase. Weber and Reinhardt (1997), using differential thermal analysis, thermal gravimetry and X-ray diffraction, observed a substantially higher amount of hydration products in LWAC at later ages, from 180 to 360 days.

Finally, since the cement paste in the LWAC does not shrink, microcracks or eigenstresses in the paste (Dela 2000), due to restraining effect of the aggregates, are avoided. This fact might also contribute to increased strength at later ages of LWAC compared to NWC.

### 3.2 Free deformations and self-induced stresses

In Fig. 3 the autogenous deformations (left) and the self-induced stresses (right) of the LWAC mixtures are shown.

The three mixes expanded from the beginning of the measurements. The expansion of the mix with Liapor F8 4-8 mm was continuous, reaching a value of 100 µstrain at 138 hours. The expansion of the mixture with the coarser Liapor F8 8-16 mm reached a peak of 85 µstrain at 30 hours. Then a small shrinkage occurred until 144 hours, followed by further expansion. At 312 hours the expansion was about 80 µstrain. The mix with Liapor sand 0-4 mm expanded rapidly until 48 hours, reaching 95 µstrain. Afterwards expansion proceeded steadily but at a lower rate until a value of 120 µstrain.

The stress development of the mix with Liapor sand showed a peak in compression (about 1.05 MPa) higher than the one observed for the mixture with 4-8 mm Liapor F8 (0.98 MPa) and for the mixture with 8-16 mm Liapor F8 (0.8 MPa). For all the mixes the peaks were reached within 24-30 hours. The relaxation of the compressive stress after the peak was more relevant for the mixture with Liapor sand (where the stresses reduce to 0.4 MPa) than for the other two mixtures. The stress development of the two mixtures with Liapor F8 was similar, showing slight relaxation until 72 hours and then an increase of the compressive stresses. In the case of the mix with 8-16 mm LWA, the experiment was prolonged to 312 hours showing continuous increase of the compressive stress, up to a value of more than 1.1 MPa. Also the mixture with Liapor sand, after relaxation of the stresses up to 132 hours, showed an increase of the compressive stresses until the end of the experiment.

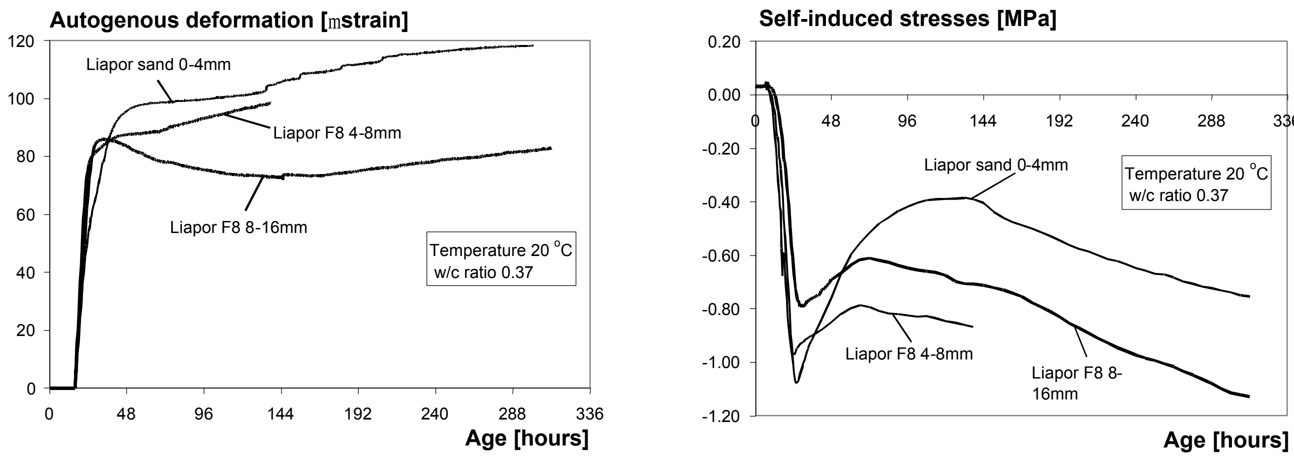



**Figure 3. Free deformation (left) and self-induced stresses (right) of mixtures with LWA of different particle size.**

# 4 EXPERIMENTS ON LIGHTWEIGHT AGGREGATES

## 4.1 General

When dealing with LWAC, one must realize that not only the mechanical properties of the aggregate (as for NWC), but also the absorption-desorption of moisture and the shrinkage of the LWA particles in consequence of moisture loss play an important role on the characteristics of the composite. These issues have been studied by means of experiments on the LWA particles and will be discussed in the following paragraphs.

Additionally, an experiment designed to visualize the transport of water from the saturated LWA to the hardening cement paste will also be presented.

## 4.2 Absorption under water

Measurements of water absorption were performed on the LWA Liapor F8, fractions 4-8 mm and 8-16 mm. The LWA were kept under water for one or two weeks and weight was measured every few days. Results are shown in Figure 4, left.

The two fractions show different absorption behaviors. The coarser fraction absorbed about 17% by weight in the first day and then the weight remained almost unchanged. The finer fraction absorbed about the same amount in the first day, but the absorption proceeded until 2 weeks after immersion, reaching a value of 25% by weight. Thus, in the finer fraction of the Liapor F8 aggregates more water can be absorbed. This fact might be beneficial if one wants to minimize the LWA content and maximize the water content of the mixture.

## 4.3 Desorption isotherm

The LWA (Liapor F8) were first immersed in water for one day and then exposed to different RH levels at 20°C until moisture equilibrium was reached. Results in the higher humidity range (until 75%) are shown in Figure 4, right. The different RH levels were obtained by storage above saturated salt solutions. Already at 100% RH a great fraction of the water content (the 'fill-up moisture' according to the definition by Nielsen, 1991) was lost. This amount, about 10% by weight of the LWA, was weakly bound by capillary forces. In fact the pores of the LWA are relatively big, having an average radius about 0.2 mm, as reported by Weber and Reinhardt (1999). The fill-up moisture is thus readily available for transport to the paste.

About a further 3% water by weight of LWA is in equilibrium at RH ranging from 100% to 94%. Also this amount of water might be absorbed by the cement paste while it self-desiccates.

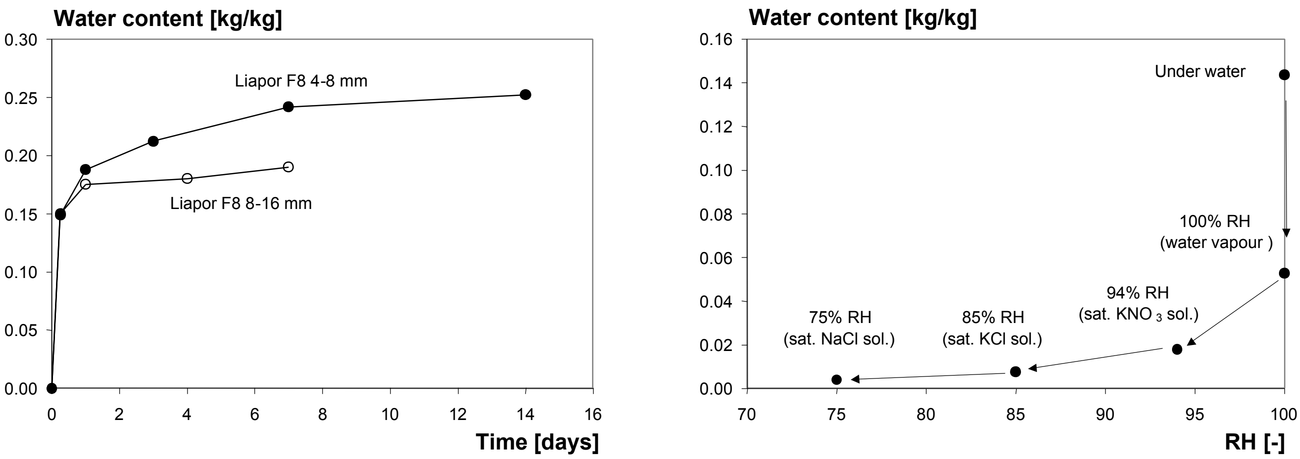



**Figure 4. Absorption under water (left) and desorption isotherm (right) of Liapor F8.**

## 4.4 Volume stability

The volume stability of Liapor F8 aggregates was tested measuring the diameter changes of single grains subjected to drying. The Liapor grains were immersed in water and then exposed to 50% RH and 20°C for several days. The weight change of one aggregate particle was measured. Additionally, LVDTs measured continuously the length changes of three Liapor grains. In Figure 5 the average result of the three parallel tests is shown, together with the weight loss.

The LWA showed substantial shrinkage. For the same LWA type, Schmidt-Döhl and Thienel (2000), using a technique based on microscopy observations, observed no clear trend of shrinkage (but possibly even some swelling) for drying from saturated conditions first to 65% and then to 44% RH. This discrepancy has no clear explanation.

Evaluation of the shrinkage measurements is complicated by the fact that evaporation of water causes a temperature drop in the LWA that results in further shrinkage. The temperature drop on the particles has been also measured and amounts to about 1 K.

This value corresponds roughly to 10 µstrain, if we assume a thermal dilation coefficient of the Liapor grains similar to the one of concrete.

Furthermore, it must be pointed out, that the shrinkage shown was found in the case of almost complete drying of the LWA (down to 50% RH). In the case of a LWAC at early ages, the relative humidity in the concrete is expected to remain high, above 95%. For these high RH levels, the LWA might be considered as non-shrinking if compared to the volume changes of the concrete.

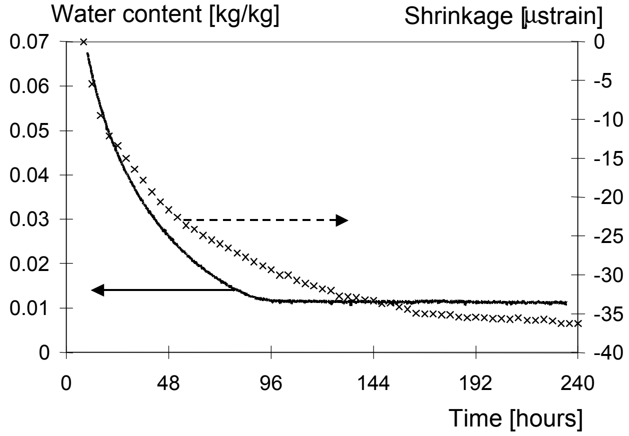
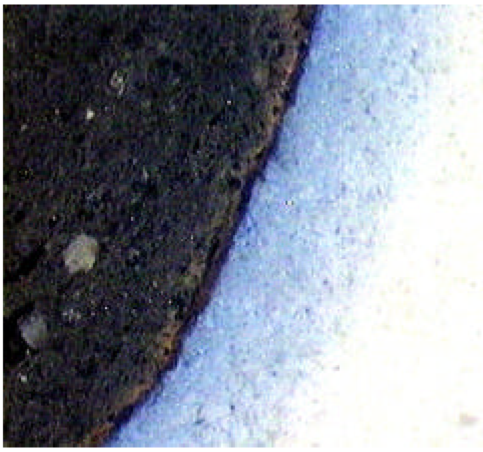



**Figure 5. Weight loss and shrinkage of wet LWA subjected to drying at 50% RH.**



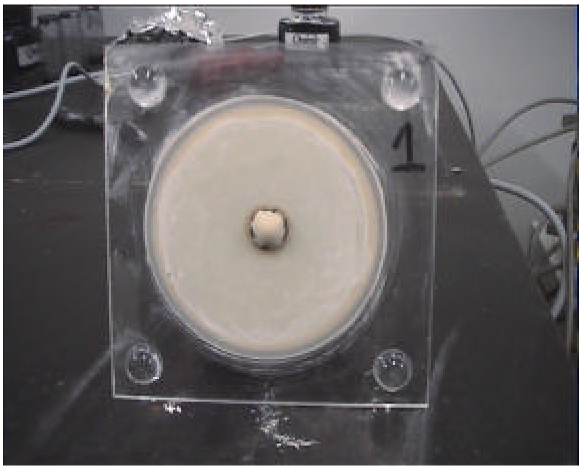


**Figure 6. Blue-ink corona around a Liapor aggregate (left), and setup to observe the growth of the corona around the Liapor grain (right).**

### 4.5 Visualization of water transport from the LWA particles to the cement paste

Liapor F8 grains were immersed in a thin blue-ink solution. The LWA's surface was subsequently dried with blotting paper and fresh cement paste was poured on them. The specimens were then sealed. In this experiment, due to a better contrast with the ink, a white Portland cement (CEM I 42.5) was used; w/c ratio of the paste was 0.3. At regular intervals, the cement paste was split. A blue ink corona (Figure 6, left) was observed around the Liapor grains. The thickness of the colored shell was growing in time, reaching 1 mm two weeks after casting.

In order to be able to follow continuously the process, the simple device shown in Figure 6 (right) was realized. The aggregate was fixed to a hole in a Perspex disc. The outer surface of the LWA was sealed with glue to avoid evaporation through the aggregate. The LWA was saturated with ink, the surface was dried and the cement paste was poured. The test was performed on three specimens at the same time. This method enabled to see the ink diffusing into the cement paste. The age at which this process occurred depended on the degree of saturation of the LWA. In one case, the diffusion of the ink was noticed two weeks after casting.

It is pointed out that the fact that ink diffuses into the cement paste does not per se mean that also water diffuses with the same speed and depth. Nevertheless, the diffusion of the ink-molecules might indicate the possibility of water-transport in the hardening paste. In this case, transport of water from the LWA to the paste in the self-curing process might be efficient on the scale of millimeters. This fact has important consequences on the choice of the LWA's size. If the water transport is active until at least 1 mm from the surface of the aggregate, relatively coarse LWA particles might be also efficient in the internal curing. Experimental evidence (see for example Fig. 3) supports this finding: Liapor F8 4-8 mm particles were sufficient to totally eliminate autogenous shrinkage in the first week.

In order to understand the self-curing process, the assessment of the distance over which water transport is effective in a concrete is of fundamental importance. Observation of ink transport into the paste gives obviously only a qualitative answer to the problem. The definitive solution might be achieved with other more complex techniques, such as X-ray absorption (see for example Bentz and Hansen 2000) or Nuclear Magnetic Resonance. Both these non-destructive techniques might be able not only to follow the transport process during hardening of the cement paste, but also to quantify the flow of water from the LWA.

## 5 MODELING THE INTERNAL-CURING PROCESS

### 5.1 General

In the following paragraphs some critical issues concerning the internal-curing process will be discussed. When one designs a LWAC mixture in order minimize or avoid autogenous shrinkage, some questions have to be answered:

- How much water must be entrained in the LWA in order to avoid self-desiccation (and thus self-desiccation shrinkage)? And how much of this water will be actually transported into the paste at high RH?
- Is this entrained water well distributed in the mixture? In other words, are the transport distances from the LWA to the matrix short enough to allow efficient internal curing?
- Which mechanisms cause the swelling of the LWAC?

These issues are not only fundamental in the mix design phase, they also constitute a starting point for the modeling of the internal-curing process as a whole.

### 5.2 Quantity of water needed for internal curing

According to Bentz and Snyder (1999), the volume of water needed to avoid self-desiccation (and thus self-desiccation shrinkage) equals the volume of chemical shrinkage of the cement paste:

$$W_{cur}=C\cdot\alpha_{max}\cdot CS \qquad (1)$$

where $W_{cur}$ water content in the LWA [kg/m$^3$]

C cement content of the mix [kg/m$^3$]

$\alpha_{max}$ maximum achievable degree of hydration

CS chemical shrinkage [kg water/kg cement hydrated]

Zhutovsky *et al.* (2001) calculated the mix composition of LWAC on the basis of this formula. The theoretical value of the entrained water was 20 kg/m$^3$. It was found that with this amount of water early-age autogenous shrinkage was not completely avoided. Some other authors had used a much higher amount in their experiments (Takada *et al.* 1998, Sickert *et al.* 1999, Lura *et al.* 2000).

Another approach to the problem of entrained water is given by Jensen (2001). The entrained water is the quantity needed for curing under saturated conditions according to Powers' model (Powers & Brownyard 1946). In this case hydration stops due to lack of space for the C-S-H gel. Thus, the hydration stops when the self-desiccation process has not yet begun. The quantity of entrained water can be calculated as (see Jensen 2001):

$$\left(\frac{w}{c}\right)_e = 0.18\cdot\left(\frac{w}{c}\right) \qquad (2)$$

According to Eq. (2) the water needed for complete curing in Zhutovsky *et al.* (2001) would be higher than the one calculated with (1), about 30 kg/m$^3$.

For other mixtures (Takada *et al.* 1998, Sickert *et al.* 1999, Lura *et al.* 2000), including the ones studied in this paper, the presence of silica fume in the mixture could lead to additional self-desiccation (Jensen & Hansen 1996); in this case, the quantity of water needed for internal curing is even more. This amount can be estimated using Powers' model modified with the addition of silica fume (Jensen & Hansen 2001, Jensen 1993). A fundamental assumption (and strong limitation) of this model is that the degree of reaction of the cement and of the silica fume is the same throughout hydration. According to this last approach, the water needed for complete curing in the mixes presented in this paper would be 47 kg/m$^3$. It is still a rough approximation, since the cement used (see Table 1) contains about 35% BFS, whose chemical shrinkage is presumably different from Portland cement. The entrained water calculated with this last approach is closer to the quantity actually stored in the LWA, especially if one considers that only about 2/3 of the water absorbed in the LWA is available for transport to the paste at high RH (as resulting from the desorption isotherm, Fig. 4, right).

### 5.3 Availability of water for internal curing – distribution of the LWA in the mixture

Sufficient water entrained in the LWA does not mean that this water will be available for transport to the self-desiccating paste. In order to obtain this, the maximum distances of the LWA particles from any point in the paste must be compatible with the distance covered by the water flow in the hardening paste.

Lu and Torquato (1992) have developed a model, based on analytical formulas, to calculate the ‘nearest neighbor’ functions in a collection of spheres of various sizes placed in a volume. The spheres are placed according to equilibrium statistics, i.e. as they were floating in a liquid without gravity and able to reach the desired position. This is a good approximation of the process happening during concrete mixing, if gravity forces are neglected. In the interpretation given by Bentz and Snyder (1999), Lu and Torquato’s model may be used to give an estimation of the total volume of matrix that is within a certain distance from a LWA. The aggregates are simulated as spheres, following the sieve analysis. Within each sieve, the aggregates are distributed by volume (Garboczi & Bentz 1997). The LWA particles are surrounded by a shell of given thickness (equal for all the spheres), representing the influence distance of the self-curing process. According to this approach, the volume of the shells can be calculated as:

$$V_{SH} = 1 - f_{LWA} - (1 - f_{LWA})\exp[(-pr(c \cdot t_{SH} + d \cdot t_{SH}^{2} + g \cdot t_{SH}^{3})] \qquad (3)$$

where $\phi_{LWA}$ is the volumetric fraction of aggregate

$t_{SH}$ is the thickness of the influence shell, equal for all the spheres

$\rho$ is the number of LWA per unit volume

c, d, g are functions of the LWA’s particle size distribution (PSD)

Equation (3) accounts for the overlapping of the spherical shells surrounding each particle. The $V_{SH}$ calculated in this way does not distinguish between cement paste and NWA that might be present in the proximity of the LWA. However, Bentz and Snyder (1999) showed, by comparison with a numerical 3D model, that the errors of this approach are limited.

In Fig. 7 the calculated cured volume, as a percentage of the total volume, is plotted against the thickness of the influence shell around the aggregates for the case of the Liapor F8 4-8 mm and Liapor F8 8-16 mm mixes. If we suppose for both mixes a shell thickness of 1 mm (in agreement with results of the ink diffusion experiments, see Fig. 6 left) around the LWA particles, for the finer aggregates (4-8 mm) the cured volume is about 70%, while for the coarser aggregates (8-16 mm) it is only about 40% (as shown by the dots in Figure 7). This approach clearly shows the beneficial effect of using finer LWA on the effectiveness of internal curing. It is pointed out that if we instead assume a maximum transport distance for the entrained water of 200 μm, as proposed by Bentz and Snyder (1999), only about 10% of the paste would be cured (see Fig. 7). The effect of the saturated LWA on the curing of the concrete would be in this case negligible. But this is in contradiction with the experimental data, since the LWAC show constant swelling (Fig. 3, left) and this is only possible if there exists a good communication between water in the LWA and the paste. In other words, the experiments (combined with results from Eq. 3) show that the water transport is efficient at a distance of at least one millimeter from the rim of the LWA.

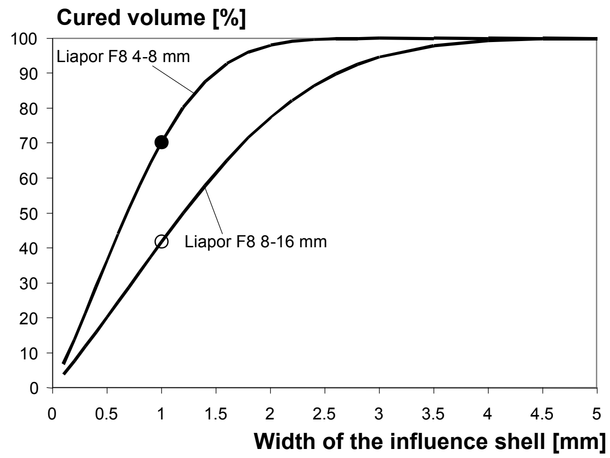



**Figure 7. Cured volume vs. thickness of the influence shell in two LWAC mixtures.**

### 5.4 Origin of the early-age swelling

Early-age swelling in LWAC was measured by a number of researchers (Takada *et al.* 1998, Sickert *et al.* 1999, Kohno *et al.* 1999, Bentur *et al.* 1999, Lura *et al.* 2000, Zhutovsky *et al.* 2001). Sickert *et al.* (1999) measured continuous swelling for more than one year. This swelling is similar to the swelling of cement paste and of NWC under water, in the case of thin specimens. In fact the situation inside the LWAC is of a cement paste hardening under (almost) saturated conditions, if the water transport from the LWA to the matrix is efficient.

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A possible explanation of the swelling could be found at the scale of the hydration products. Even if the reaction products have a lower volume (about 7% less) than the reagents, due to their shape they form a spatial network. Growth of further reaction products inside the network generates an internal pressure that may cause moderate swelling of the system (Bažant & Wittmann 1982, Schmidt-Döhl & Rostásy 1995).

Budnikov and Strelkov (1966) proposed a slightly different mechanism. During hydration, a cement particle is converted in a number of much smaller hydrated particles through topochemical reactions. These smaller reaction products tend to occupy a larger volume than the unhydrated particle, generating an internal pressure that produces macroscopic swelling.

## 6 DISCUSSION AND CONCLUSIONS

The incorporation of water-entrained LWA in a concrete mixture is a very efficient way to reduce autogenous shrinkage in High-Performance Concrete. Low w/c ratio LWAC mixtures with saturated aggregates show generally swelling in the early phase of hardening, due to a moisture flow from the LWA to the cement paste. This phenomenon is extremely complex and poorly understood.

The most relevant findings for the concretes and the LWA considered in this paper are the following:

- The strength of LWAC showed an increase at later ages, if compared to NWC with the same matrix. This fact might be due to further hydration promoted by the water entrained in the LWA.
- Mixtures with small and homogeneously distributed saturated LWA showed more swelling (in the first two weeks after casting) and higher self-induced compressive stresses than mixtures with coarse aggregates.
- Not all the water absorbed in the LWA (which might be as high as 25% by weight, after several days under water) is available for transport to the self-desiccating paste. About 6% water (by weight of the LWA) will remain in the LWA at 100% RH. This value drops to 2% at 94% RH.
- Water-saturated LWA shrunk when exposed to 50% RH, as a consequence of the moisture loss. This shrinkage is difficult to separate from thermal contraction due to the temperature drop that accompanies evaporation. Furthermore, it is believed that, if the RH-drop in the paste is limited, the shrinkage of the LWA will be only minor.
- In the hardening cement paste around LWA particles saturated with a thin ink-solution an almost spherical colored shell expanding with time was visible. After two weeks a depth of about 1 mm was measured. This indicates the potential for moisture transport in the hardening paste.
- Analytical calculations, based on the PSD of the LWA particles, show that (at least for the concrete containing the finer fraction Liapor F8 4-8 mm) most of the cement paste is influenced by the internal curing process, if we assume a penetration depth of the entrained water of about 1 mm. According to these results, gravel-size LWA might be adequate enough to provide uniform internal curing of the composite.

As a final remark, it must be emphasized that the early-age behavior of LWAC is a result of a complex interaction between mechanical, transport and deformational properties of both the LWA and of the hardening paste. A thorough knowledge of the behavior of both materials is a prerequisite to proceed towards modeling of the LWAC as a whole.

## 7 ACKNOWLEDGMENTS

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# Durability Of Externally Bonded FRP Systems For The Strengthening Of Existing Structures

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**Summary: Fibre Reinforced Polymer (FRP) systems are increasingly being used in the strengthening of civil structures. It is generally believed that they have a very high level of durability. However, this is not necessarily true and depends on the specifics of the materials chosen, the processing technique used, the type of existing load regime and level of environmental exposure.**

**The purpose of the research (in order to give some guidance to engineers when selecting FRP for construction) is to present a study on the application of a general method, developed by Nicolella[1], to determine the Estimated Service Life (ESL) of *Externally Bonded* FRP *Reinforcement (EBR)* systems used for the strengthening of existing structures.**

**The paper provides an overview of degradation mechanisms, under the effect of the most influential agents, with reference to internationally published research, in order to develop a knowledge base to allow the implementation of the method.**

**The result is the definition of the agents which, according to the method, influence the service life of an FRP EBR system and that lead to the determination of modifying factors of a mid-normal value, derived from field-collected data for assumed conditions[2].**

**Keywords: Fibre Reinforced Polymers, Durability, Influencing Agents, Method of M. Nicolella.**

## 1 INTRODUCTION

The use of externally bonded FRP systems has emerged all over the world as an alternative to conventional strengthening techniques and features various advantages. Although numerous studies and experiments aimed at defining mechanical and physical properties have been carried out along with the development of structural models and calculation methods, the durability of these materials in service conditions is still an open matter.

For evaluating the durability characteristics of FRP reinforcing systems, accelerated aging experiments have been performed world wide, but it is very difficult to compare and hence difficult to interpret the research results due to 2 different aspects:

1. there are no international standard FRP durability test methods;
2. there are too many differences in production methods and in material systems among the manufacturers;

Thus, testing procedures should be standardized.

Durability is the capability of a building or its parts to perform its required function over a specified period of time under the influence of the agents anticipated in service.

The required performances for externally bonded FRP systems are: the mechanical properties (tensile strength, elastic moduli, etc.) and the bonded capacity to existing surfaces. These performances degrade not only under exposure to certain agents, such as alkalinity, acidity, salt water, moisture, ultraviolet light, high temperature and freezing and thawing cycles, but as an effect of other factors such as production technologies and the work execution. Experimental research does not generally take into account these other factors.

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[1] Professor Maurizio Nicolella[1] of the D.IN.E.(Department of Building Engineering of Naples – ITALY)

[2] For a complete presentation of the method there is the article of M. Nicolella, *Components Service Life: from field test to methodological hypothesis.* For the comparison with the Factorial Method proposed by ISO 15686 there is the article of A. De Pascale *Modifying factors of a method for estimating service life,* both published in the same proceeding of the Congress.

In order to understand the consideration expressed below, it is necessary to explain the basic FRP EBR (*Fibre Reinforced Plastics Externally Bonded Reinforcement)* strengthening technique and their failure mechanisms. Two techniques are generally used: *wet lay up systems* and *pre – cured systems.*

In the *wet lay-up* FRP system, non-directional or multidirectional dry fibre sheets or fabrics are saturated with resin and treated on the work site. Whilst, the *pre-cured* FRP systems consist of a variety of composite shapes manufactured off-site and transferred to the work site. Typically, an adhesive along with the primer and putty is used to bond the pre-cured shapes to the application surface. Clearly these two different techniques influence the long-term behaviour of the strengthening composite. In any case, three main elements, which influence the long-term behaviour, can be highlighted: the matrix, the fibres and the interface FRP –application surface (typically concrete or masonry). This interface is the critical component for the effectiveness of most externally bonded FRP structural strengthening applications, as it is the location where the transfer of stresses occurs.  Field experience has shown that the bond quality is influenced by a lot of factors such as: the condition of the existing surface, its preparation, the quality of the FRP application, the quality of the FRP itself and the resin. Also internal and external factors play a fundamental role in the long-term behaviour of the combined system. The figure below shows all factors, which should be taken into consideration in order to predict and to guarantee FRPs durability, as shown in Figure 1

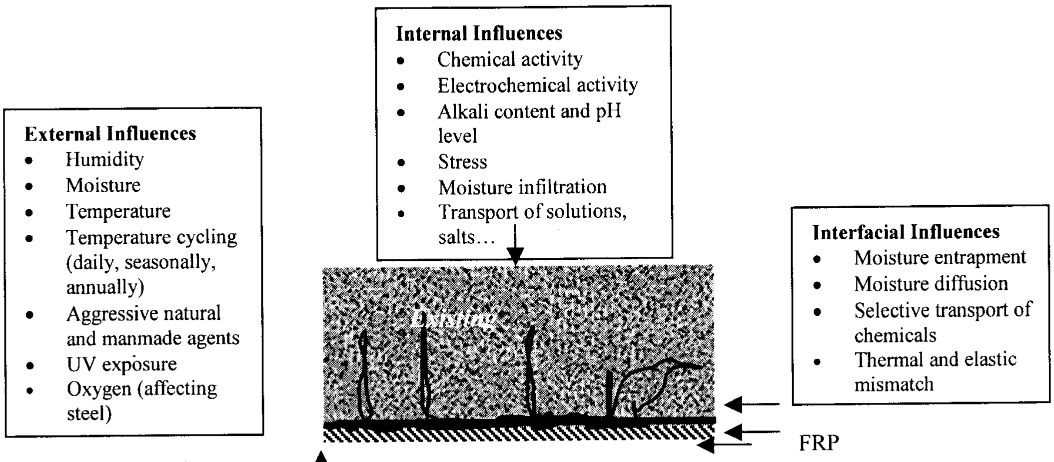


**Figure 1. FRP system level interactions** (fibTG9.3- Working group «Externally bonded reinforcement» - EBR 'Special design considerations & environmental effects')

The typical damages, which can affect the interface, are: *debonding or peeling.* The *debonding* is the separation at the interface between the application surface and the reinforcing layer, while the *peeling* is the separation at the interface between the substrate and the reinforcing layer.

In addition to performance decline of the interface, other pathologies can affect the FRP system. As a consequence of composites heterogeneity, anisotropy and lack of ductility, it is very difficult to provide the type of collapse under the effects of the influencing agents and their combination. Failure can occur for each one of the main mechanisms below:

- matrix yielding;
- *buckling* of fibres;
- *pull out and prominence* of fibres;
- *delamination;*
- Fibre yielding.

*Delamination* is the separation along a plane parallel to the surface, as in the separation of the layers of the FRP laminate from each other, but also the separation of the individual fibres from the matrix, which can affect performance.  This phenomenon occurs when matrix and interlaminar shear strength is low and fibre stiffness is low too. *Fibre yielding* occurs when there are manufacturing defects, when different fibres haven't the same resistance, or the matrix in unable to distribute the loads among the fibres. *Pull out* occurs when broken fibres leak from the matrix. This happens when the fibre – matrix link is short, and also depends on the transfer loads mechanism among fibres and matrix; *Prominence* occurs when the matrix degrades exposing the fibres, while *"buckling"* of the fibres is the failure due to instability of the compressed fibres.

## 2 AGENTS CAUSING THE DEGRADATION OF EXTERNALLY BONDED FRP SYSTEMS AND THEIR EFFECTS - STATE OF THE ART -

Many factors play an important role in reducing a building component's life, which in most cases, is very difficult to define or estimate. Therefore, it is advisable that, before using FRP composites, possible changes that may occur, under given service conditions, should be known.

The behaviour in service conditions and, in particular, the service life of a component are influenced by the agents that cause degradation, by their intensity, their variations and their combination.

In this part of the paper, durability related aspects of FRP strengthening systems, studied by several authors, will be briefly discussed to demonstrate that current results are too confused. The exposed results regard studies on the externally bonded FRP systems for the strengthening of concrete structures under some of the most influential agents.

Such agents will be taken into account in the estimate of a component's service life in the method explained below.

## 2.1 Climatic agents

### 2.1.1 Temperature and temperature cycling

It is well known that high and low temperatures or temperature cycling (daily, seasonally, annually) influence FRP strengthening systems failure process. The effect of the high temperatures on resin matrix is to reduce the stiffness; consequently the matrix decreases its ability to distribute the loads among the fibres and to protect them. The loading on individual fibres may exceed the capacity of the fibre, which will collapse. The next fibre picks up the load and, if the stress level is too high, it will also collapse and so on. For hight temperature, the threshold value is the so called "Glass Transition Temperature" (Tg) defined as *the midpoint of the temperature range beyond which the resin component of a fibre reinforced thermosetting polymer material changes from a brittle state to a ductile state* (def. ACI 440F). Thermal energy adsorbed over glass transition temperature can cause ultimate load carrying capacity of the FRP lower in measure of 30-40% than the initial value (Kelley et al. 1999).

The thermal cycling had a greater effect. It has been shown that thermal cycling causes debonding between the resin and the fibre. This deterioration may occur when constituents have different coefficients of thermal expansion. Particularly transverse thermal expansion is important for a good bond (Sen et al., 1999).

Low temperatures cause internal stresses in the composite, therefore causing matrix stiffness to increase. Low temperature cycling (+50°C/-60°C) influences both E and G moduli of the composite (its mechanical properties). The resulting degradation appears to be originated in the matrix, mainly due to microcrack propagation through the resin. Very low temperatures (lower then 0°C) and moisture can cause the freezing of the water in the existing cracks or voids, consequently the volumetric expansion of the freezing water causes the debonding of the FRP. Tests (Pantuso et al. 2000) have shown that specimens tested to failure under low or high temperature show not only variations in the ultimate bond force, but also significant differences in the nature of debonding (bond failure in the thickness of the glue or peeling for the specimens with high modulus of elasticity).

### 2.1.2 Freeze-Thaw

Often, when a FRP strengthening system is applied on existing structures, these structures are still cracked. The bonding of the FRP on the application surface may not be perfect and some voids or delaminated areas may be present in the interface layer. As a consequence, the expansion of freezing water in these cracks or voids may cause debonding, peeling or delamination of the FRP.

Tests on reduced - scale concrete beams, inferiorly plated with FRP (epoxy matrix), have shown significant deterioration in ultimate strength and partial degradation ("debonding") at the FRP – concrete interface, when subjected to freeze – thaw (-17°C - ambient temperature) cycles. This degradation has only contributed CFRP (*Carbon Fibre Reinforced Polymers)* specimen failure. Moreover CFRP platings seem to be more durable than GFRP ones. After 100 freeze-thaw cycles, in fact, the strength of the CFRP plated beams is over 130% that of non-plated beams (subject to the same conditions), while in the case of the GFRP (*Glass Fibre Reinforced Polymers)* plating the increase is below 70%. (table 1 and 2, Green M. et al. 1998),

**Table 1. Failure modes**

| STRENGTHENING TYPE | FAILURE MODES AFTER 100 CYCLES |
|---|---|
| NO plated | ductile flexural failure |
| GFRP plating | Tensile failure of the FRP laminate |
| CFRP plating | Tensile filure of the FRP laminate and debonding |

**Table 2. Environment effects on tensile strenght increase (%)**

| STRENGTHENING TYPE | BEFORE CYCLES | AFTER 100 CYCLES |
|---|---|---|
| GFRP | 88 | 67 |
| CFRP | 139 | 131 |

Concrete specimens plated with FRP subjected to a uniform tensile stress and to 300 freeze / thaw cycles have shown no significant influence on the value of the tensile strength compared to the behaviour of reference elements. Test results show that specimens wrapped by CFRP have not endured lessening of tensile strength, while those wrapped by GFRP show a reduction of about 12% (Dutta P.K. and Hui D., 1998) It is possible to conclude that repeated freeze-thaw cycles have shown little or no effect on the FRP when high quality, moisture resistant epoxies are used.

### 2.1.3 UV light exposure

Ultraviolet rays affect polymeric materials. UV radiation can cause dissociation of chemical bonds. Sunlight and especially ultraviolet light can lead to a reduction of light transmissibility and colour changes in the composite. The resin matrix materials are generally susceptible to UV light damage, especially for polyester resins more than epoxy resins, although they show somewhat similar strength reduction. This reduction can cause *cracking* or *crazing* (fine cracks at or under the surface) of the matrix of the composites. This may lead to other environmental problems, such as increased moisture absorption and/or chemical attack.

Some specimens tested in a laboratory environment for 250, 750 and 1250 wetting/drying cycles, with UV- intensity of 0,2 $MJ/m^2$/hour and temperature of 26°C, have shown these results:

AFRP (*Aramid Fibre Reinforced Polymers)* rods showed around 13% reduction in tensile strength after 2500 hours exposure; GFRP rods 8% after 500 hours (no reduction thereafter); CFRP rods showed no reduction (Kato et al. 1997).In field tests, AFRP have shown to be more susceptible to UV than CFRP and GFRP, in accordance with experimental results (Bank and Gentry 1995).

In general, carbon fibres are substantially unaffected by UV light. Glass fibres are only slightly affected by UV light, while Aramid is the most affected by UV. However the amount of deterioration is dependent on the type of resin and the fibre type (Ahmad and Plecnik 1989).

### 2.1.4 Moisture

The presence of moisture in the composites is caused by the mechanism below:

- Capillary action along the longitudinal axis of the fibre or at the resin-fibre interface;
- Transfer through cracks and voids in the structure;
- Diffusion through the matrix.

The water absorption on the matrix, whose amount is dependent on the resin type and the water temperature, causes the reduction of the glass transition temperature and the stiffening of the resin. Both of these effects are partially reversible in epoxy resin when the water is removed by drying. With polyesters and vinyl esters the changes can be reversible or not, depending on the time and temperature of the exposure. (Blaschko et al. 1998). According to other authors, the moisture would have been turned into steam, causing voiding well before the Tg is reached (Burn 1997).

Damage to fibreglass/epoxy composites may be caused by the intrusion of moisture into the resin-fibre interface. Aramid fibres absorb up to 13% by weight of moisture that can have a deleterious effect on tensile strength and can affect the resin-fibre interface. Carbon fibre is relatively inert to water and so the only effects on CFRP are those of moisture on the resin matrix. (*Fib* bulletin, 2001)

In order to estimate the effect of the moisture on the tensile strength, some concrete specimen wrapped with FRP are subjected to 300 humid/dry cycling. While CFRP strengthened specimens haven’t shown any damage the GFRP ones have shown a reduction of 20%.

Moreover the CFRP strengthened beams seem to be more durable than GFRP one; after 100 humid/ dry cycles, in fact, the resistance of the beams plated with CFRP is over 120% compared with the non strengthened ones (subject to the same conditions), while in the case of GFRP strengthening the increase is below 40% (table 3 and 4) .The influence of the type of used epoxy resin becomes very marked on the value of the ultimate load of the reinforced beam. Tests on FRP wrapped columns haven’t shown a reduction of strength and ductility, while the columns wrapped with GFRP have shown a little reduction when some adhesive has been used (Green M. et al. 1998).

**Table3. Failure modes**

| STRENGTHENING TYPE | FAILURE MODES AFTER 100 CYCLES |
|---|---|
| NO wrapped | Ductile flexural failure |
| GFRP wrapped | Tensile failure of the FRP laminate |
| CFRP wrapped | Tensile filure of the FRP laminate and debonding |

**Table 4. environment effects on tensile strenght increase (%).**

| STRENGTHENING TYPE | BEFORE CYCLES | AFTER 100 CYCLES |
|---|---|---|
| GFRP | 88 | 37 |
| CFRP | 139 | 120 |

## 2.2 Environmental agents

### 2.2.1 Alkalinity/Acidity

The performance decay of the FRP strengthening over time in an alkaline or acid environment will depend on both matrix and the reinforcing fibre. Carbon fibre is resistant to alkali and acid environment whereas glass fibre can degrade in these environments. Aramid fibres are affected by acid environment and present good resistance to alkali attack. However a properly applied resin matrix will isolate and protect the fibre and postpone the deterioration. The chemical resistance of different types of fibres is given in table 5

**Table 5. Qualitative assessment of fibres with respect to chemical resistance. Notations: A=excellent, B=good, C=passable and D=poor. (Table found in fib bulletin)**

| 2.2.2 Environments | | Carbon fibre | | | | Aramid fibre | | Glass fibre | |
|---|---|---|---|---|---|---|---|---|---|
| | | GP-grade pitch | HP-grade pitch | HT-type PAN | HM-type PAN | Kevlar -49 | Tecchnora | E-glass | AR-Glass |
| **Acid Resistance** | Hydrochloric acid | B | A | A | A | D | B | D | - |
| | Sulfric acid | A | A | A | A | D | B | D | - |
| | Nitric acid | B | A | A | A | D | B | D | - |
| **Alkali Resistance** | Sodium hydroxide | A | A | A | A | B | B | C | B |
| | Brine resistance | A | A | A | A | B | B | C | - |
| **Organic Solvent resistance** | Acetone | A | A | A | A | A | | A | - |
| | Benzene | A | A | A | A | A | B | A | |
| | Gasoline | A | A | A | A | A | B | A | - |

### 2.2.3 Salt

Some structures can be exposed to salts from the sea (coastal structures) or in the form of de-icing salts (bridges in cold regions). There are several results indicating that the saline solution is more severe than fresh water. (Saadamanesh and Tannous 1997; Steckel et al 1998; Gangarao, H. V. S. and Vijay, P. V.; 1997 Sasaki, I. Nishizaki, I. et all 1997). Investigations on GFRP structural composites under salt exposure (bars and plates, concrete beams reinforced with GFRP and wrapped with GFRP fabrics) have shown a reduction in strength and stiffness up to 50%, which correspond to 10- 50 years of natural exposure, depending on the severity of environment and temperature CFRP's performances are superior compared also to AFRP.

### 2.2.4 Fatigue, creep behaviour, stress rupture and stress corrosion

The behaviour under conditions of sustained loads or fatigue loadings is another topic related to the durability of FRP strengthened structures. CFRP, exhibit superior fatigue performance to that of steel. Some investigations have shown that the fatigue performances of concrete beams can be enhanced significantly by means of FRP strengthening (*fib* bulletin, 2001).

The concrete elements exhibit varying degrees of creep deformation whilst under constant load CFRP does not creep, the creep of GFRP is negligible, but that of AFRP cannot be neglected. Hence, the creep behaviour of CFRP- or GFRP-plated RC elements is governed, primarily, by the compressive creep of concrete (e.g. Plevris and Triantafillou 1994). As AFRP creeps itself, long-term deformations increase considerably in the case of AFRP-strengthened elements.

Other important topics related to durability of externally bonded FRP systems are the stress rupture and the stress corrosion. The first one is the premature tensile rupture under sustained stress. Indeed stress corrosion occurs when the atmosphere or ambient environment is of a corrosive nature, but not sufficiently so that corrosion would occur without the addition of stress. Failure is deemed to be premature since the FRP fails at a stress level below its ultimate. In general, the following order of fibres and resins gives increasing vulnerability either stress rupture or stress corrosion (Kelley et al. 1999): carbon-epoxy, aramid vinyl ester, glass polyester. In general, given the stress rupture of GFRP and the relatively poor creep behaviour of

AFRP, it is recommended that when the externally bonded reinforcement is to carry considerable sustained load, composites with carbon fibres should be the designer’s first choice (*fib* bulletin, 2001).

## 3 DURABILITY APPROACH IN EXISTING DESIGN GUIDELINES

The properties of FRP reported by the manufacturer (based on testing conducted in a laboratory environment) do not reflect the effects of environmental exposure. The long-term exposure to various types of environments can reduce the tensile properties, creep-rupture and fatigue endurance of FRP laminates. In order to consider this matter the existing guidelines propose to reduce the value of mechanical properties in design equations, using environmental reduction factors given for the appropriate fibre type and exposure condition.

| Factor | ACI | NS3473 | CHBDC | JSCE | BISE |
|---|---|---|---|---|---|
| Reduction due to **environmentally** caused deterioration | $C_E$ "environmental reduction factor" GFRP: 0.70-0.80 AFRP: 0.80-0.90 CFRP: 0.90-1.00 | $\eta_{env}$ "conversion factor" GFRP: 0.50 AFRP: 0.90 CFRP: 1.00 | * $\phi_{FRP}$ "resistance factor" GFRP: 0.75 AFRP: 0.85 CFRP: 0.85 | ** $1/\gamma_{fm}$ "material factor" GFRP: 0.77 AFRP: 0.87 CFRP: 0.87 | $1/\gamma_m$ "material factor" GFRP: 0.30 AFRP: 0.50 CFRP: 0.60 |
| Reduction due to **sustained stress** | | $\eta_{lt}$ "conversion factor" GFRP: 0.8-1.0 AFRP: 0.7-1.0 CFRP: 0.9-1.0 | F "factor" GFRP: 0.8-1.0 AFRP: 0.5-1.0 CFRP: 0.9-1.0 | | |
| **Total strength reduction** due to environment and sustained stress | GFRP: 0.70-0.80 AFRP: 0.80-0.90 CFRP: 0.90-1.00 | GFRP: 0.40-0.50 AFRP: 0.63-0.90 CFRP: 0.90-1.00 | GFRP: 0.60-0.75 AFRP: 0.42-0.85 CFRP: 0.76-0.85 | GFRP: 0.77 AFRP: 0.87 CFRP: 0.87 | GFRP: 0.30 AFRP: 0.50 CFRP: 0.60 |
| Specified **stress limits** for permanent load | GFRP: 0.14-0.16 AFRP: 0.16-0.18 CFRP: 0.44-0.50 | Stress limits not specified | GFRP: 0.60-0.75 AFRP: 0.42-0.85 CFRP: 0.76-0.85 | 0.8 × "creep failure strength" not more than 0.7 GFRP: ≤ 0.7 AFRP: ≤ 0.7 CFRP: ≤ 0.7 | Stress limits not specified |

**Table 6. Reduction factors used in existing design guidelines (JSCE 1997, CHBDC 1996, ACI 2000, Thorenfeldt 1998 and Clarke et al 1996)x ( Byars et al. 1st draft of the Fib bulletin TG 9.3 on Durability of FRP)**

Some design guidelines incorporates several different types of strength reductions, including those caused by environmental degradation, i.e. ACI and JSCE, while for example NS3473 has one specific factor for taking account of environmentally caused deterioration (table 6).

The existing design guidelines do not give a method to estimate the service life of an FRP system.

## 4 THE APPLICATION OF THE METHOD PROPOSED BY MAURIZIO NICOLELLA TO DETERMINE EXTERNALLY BONDED FRP SYSTEMS SERVICE LIFE

The research group of the author is carrying out a study on the application of the method by Nicolella (Nicolella M., *Affidabilità e durabilità degli elementi costruttivi in edilizia – Un‘ipotesi metodologica per il calcolo,* CUEN, Napoli, 2000) to determine the ESL (Estimated Service Life) of FRP externally bonded systems (figure 2).

The most difficult issue regarding durability studies is taking into account all factors (environmental, mechanics, etc.) that affect the service life of a building or its components. This method provides a simplified procedure for considering each of the variables that are likely to affect service life.

The methodology estimates the service life of a component as a deviation from a standard value, named *mid-normal*, which is calculated by using *modifying factors* deriving from *influencing agents.* The *“mid-normal”* value is obtained through statistical elaboration of field-collected data for the assumed conditions.

According to that method, the phases needed to determine the service life of a building component are listed below:

3. Definition of the experimentation on the field, in order to calculate all the parameters of the *"mid- normal" value;*
3. Definition of the agents mainly influencing the service life of the chosen component;
3. Dimensioning of the modifying factors;
3. Validation of the adopted factors through laboratory tests.

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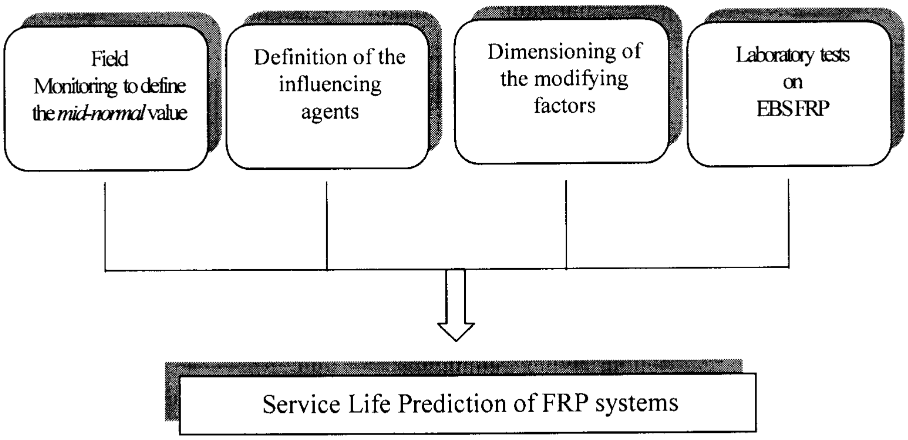


**Figure 2.** **Schematic description of the study program of our research group**

In order to assess the durability of a component it is necessary to fix performance levels, under which the inquired component has to be considered in state of damage. In the present study, the failure is estimated in terms of performance decay of bonding capacity and strength.

In order to identify the failure due to peeling, delamination or debonding, reference will be done to the permissible threshold values (in terms of percentage of degraded surface with respect to the entire plated surface), as indicated in ACI 440 – F (2000); in order to asses tensile strength degradation, creep – rupture and fatigue performance of FRP laminates, the value of the residual strength will be compared with those calculated according to existing guidelines.

It is also necessary to observe that, for each building component, also intermediate performance levels are to be considered. Table 7 shows intermediate performance levels along with the relevant intervention classes.

**Table 7. The intermediate performance**

| DEGRADATION | | PERFORMANCE | | INTERVENTION TYPOLOGIES |
|---|---|---|---|---|
| State1 | **?** | Level 1 | **?** | Monitoring /inspection |
| State2 | **?** | Level 2 | **?** | Cleaning / repair of surface |
| State3 | **?** | Level 3 | **?** | Repair/restoration |
| State4 | **?** | Level 4 | **?** | Partial substitution / integration |
| State5 | **?** | Level 5 | **?** | Total substitution |

The degradation states and the consequent intervention typologies are specified for the FRP systems, as expressed below:

- Monitoring /inspection
  Is required for defining the in service behaviour and the sub -condition maintenance strategy. For the installed FRP, inspections can be carried out by means of visual inspection, searching for changes in colour, *debonding*, *peeling*, *blistering*, *cracking*, *crazing*, deflections, indications of reinforcing bar corrosion, and other anomalies. In addition, ultra- sonic, acoustic sounding (hammer tap), or thermographic tests may indicate and quantify the entity of possible *peeling*, *debonding* or *delamination*. A mechanical inspection, instead, can include conventional structural loading tests or pull - off tension tests.
- Cleaning / repair of surface
  The surface protective coating (plasters, varnishes, panels) is to be replaced. The surface coating may be replaced using a process approved by the system manufacturer.
- Repair/ restoration
  The intervention is aimed at eliminating the small anomalies with the respect to the restoration of the initial conditions. The smaller delaminations can be repaired by epoxy resin injection.
- Partial substitution / integration
  In this case, a part of the system is removed or replaced because the removal of the defects is not possible without adding new parts. For the FRP systems, this happens when the damage can affect the structural integrity of the laminate, such as localized FRP laminate cracking or abrasions.

This type of damage can be repaired by bonding FRP patches on to the damaged area. The FRP patches should have the same characteristics (such as thickness, ply orientation) as the original laminate and the FRP patches should be installed in accordance with the material manufacturer's recommendation.

- Total substitution
  It coincides with the *" death"* of the component and therefore it identifies its life cycle. In the case of the FRP systems this phase is identified with the peeling and/or the debonding of large areas, which require removal of the affected area, reconditioning of the substrate, and replacing the FRP laminate. With reference to the American ACI 440-F (2000) guidelines the states of degradation are therefore characterized in table 8:

**Table 8. States of degradation according to ACI 440-F**

| | |
|---|---|
| State1 | Conditions similar to initial ones, absence of superficial anomalies and patina |
| State2 | Chromatic alterations (colour changes), presence of patina, gap presence in the protective layer or small delaminations peeling or debonding presence less than 2 $in^2$each (1300 mmq) so long as the delaminated area is less than 5 % of the total laminate area and no more than 10 such delaminations per 10 $ft^2$ (1 mq |
| State 3 | Delaminations peeling or debonding presence less than 25 $in^2$ (16000 mmq) |
| State 4 | Large delaminations, greater than 25 $in^2$ (16000 mmq) but confined. |
| State 5 | Large areas subjected to peeling and/or the debonding which require removal of the affected area, reconditioning of the substrate, and replacing the FRP laminate |

According to the method, the *mid-normal* values can be applied to any other context, when manipulated through the modifying factors, which are thought to be function of the specific case.

For opportunity reasons, in order to extrapolate the mid – normal values, some FRP EBR applications carried out in Naples could be taken into account. These applications feature

- Flexural reinforcement of concrete structures;
- Plating of natural stone masonry vaults;

The choice of the models has been done privileging the following criteria:

- Quality and amount of the information available: application year, materials used, work execution, possible anomalies found;
- Possibility to execute inspections: visual controls, ultra- sonic, acoustic sounding (hammer tap), thermographic tests, mechanical tests;
- Possibility to compare materials used, serviceability of the support, geometric conformation of the application surface, etc.;
- Homogeneity of the influence agents;
- Possibility for the influence agents to vary, in order to correctly estimate their weight.

As a result of the monitoring of these applications, some performance/time diagrams will be extrapolated.

Once the *mid-normal* value has been determined, the service life of any FRP system will be evaluated through the expression (1)

$$D_{pp} = D_{mn} \times \prod_{i}^{n} F_i \qquad (1)$$

where *Dpp* is *" the most probable "* service life, *Dmn* the mid-*normal* duration and *Fi* the modifying factors expressed as a function of the groups of the influencing agents.

All the most influential agents (with respect to component life) are arranged in groups each one generating a single factor.

In brief, different "scores", depending on the real condition of the component, are assigned to each influential agent. Elaborating the "scores" regarding any group of agents it is possible to obtain the value of the modifying factors (each one weighted depending on its incidence) used in the formula. The groups of homogenous agents are those in table 9:

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**Table 9. Influencing agents**

| n | GROUPS OF AGENTS | INFLUENCING AGENTS | WEIGHT |
|---|---|---|---|
| **1** | **Technological characteristic** | System components, application surface state, installation procedure, work execution level, protection | 2 |
| **2** | **Climatic agents** | Temperature, wind and rain, snow, UV exposure, Temperature cycling (daily, seasonally, annually), Humidity and moisture | 4 |
| **3** | **Environmental/ operational environment agents** | Chemical agents, exposure to salts, in use condition (operational environment agent) | 4 |
| **4** | **Configuration** | shape/lying, extension, presence of discontinuity / chines | 1 |

1 *Technological characteristics*

1.1 The variability conditions of the *system components* agent depend on**:** resin types, fibre types, adhesive quality, *primer* chosen with respect to design parameters (environmental, economic, temporal etc.); It is favourable to define classes of variability of the "scores" to attribute, with respect to various possible connections fibre/matrix/adhesive, typically used in the FRP strengthening systems;

1.2 The classes of variability of the agent *application surface* will depend on the material and the serviceability of the reinforced surface (i.e. compact or cracked concrete, compact or cracked masonry);

1.3 The agent *installation procedure* will feature the classes of variation based on the used technology: *hand lay-up*, *automatic*, *fast curing*, *by vacuum.*

1.4 The agent work *execution level* represents the level of skill and control in site work. It is based on whether the site work is likely to be in accordance with manufacturers recommendations and highly controlled. To define the variability conditions of this agent it is necessary to verify:

- if FRP system constituent materials are packaged and shipped in a manner that complies to all applicable packaging and shipping codes and regulations;
- if the materials are stored in accordance with the manufacturer's recommendations;
- if The Shelf life is defined (the properties of the uncured resin components can change with time, temperature, or humidity). Such conditions can affect the reactivity of the mixed system and the uncured and cured properties;
- if MSDS for all FRP constituent materials and components are obtained from the manufacturers and are accessible at the work site;
- that Procedures for installing FRP systems have been developed by the system manufacturers;
- if the FRP system installation contractor demonstrates ability for surface preparation and application of the FRP system to be installed (contractor ability can be demonstrated by providing evidence of training and documentation of related work previously completed by the contractor).

1.5 The agent *protection* will feature conditions of variation depending on:

- use of protecting varnish, plasters or panels which can offer an effective protection to the fire, to UV lights etc.;
- planning of a disposition of external bonded FRP system, which doesn't determine a waterproof closing of the strengthened element, avoiding the formation of a moisture barrier;
- installation of the FRP system in areas where it is not exposed to impacts or rain.

**2.** *Climatic agents*

2.1. The conditions of variation of the agent *temperature* are due to different types of fibre/matrix/adhesive coupling. The glass transition temperature is certainly one of the thresholds of the classes of variability of this agent (possible classes of variability of the agent temperature are the following: from –20°C to 0°C, from 0°C to 60°C, from 60°C to Tg and over Tg. This latter case is possible when these strengthening systems are used in special applications, i.e. in factory).

2.2. The range of variability of the agent *rain* will be derived from the zonings introduced by national codes

2.3. The range of variability of the agent *wind* will be derived from the zonings introduced by national codes (in Italy, D.M. the 16/01/1997);

2.4. The range of variability of the agent *snow* will be derived from the zonings introduced by national codes (in Italy, D.M. the 16/01/1997);

2.5. The range of variability of the agent *UV exposure* will depend on the resin and fibre types. Certainly lower "scores" will be attributed to AFRP composites with respect to GFRP or CFRP composites (the "scores" are inversely proportional to the capacity of the agent to cause unreliability). In fact AFRP have shown to be more susceptible to ultraviolet rays than CFRP and GFRP;

2.6. The range of variability of the agent *daily temperature cycling* will be derived from local indications;

2.7. A maximum water uptake value of 3% of the weight is normally specified for structural adhesives (Blaschko et al. 1998). Hence, in order to define the range of variability of the agent *moisture* it is possible to fix two or more ranges, whose thresholds could be the 3% of the weight;

3. *Environmental* **-** *operational environment agents*

3.1 The range of variability of the agent *chemical agents* will be obtained defining the different types of chemical agent (which affect the service life of the component) and the different types of resin/matrix/adhesive system;
3.2 The FRP strengthening systems could be exposed to salts from the sea (costal structures) or in the form of de-icing salts; both the variability conditions and classes of the *exposure to salts* should be defined on the basis of the closeness to the see or the use of de – icing salts;
3.3 The agent *in use condition* reflects the effect of the use condition of the component Regarding FRP systems the in-use conditions could involve phenomena such as fatigue, creep behaviour, stress rupture and stress corrosion. The classes of variability of the “scores” to assign to this agent should be defined taking into account the behaviour of the various systems under these conditions.

4 *Configuration*

4.1 the agent *shape/lying* will comprise the following classes of “scores” variation: plane surface, concave surface, convex surface;
4.2 the agent *presence of discontinuity and/or chines* will be able to preview classes of variation based on the incidence of such presences on the area of the reinforced surface;
4.3 the agent *extension* will depend on the value of the reinforced surface compared with the un-reinforced one.

As far as the application of the method to the EBR FRP systems is specifically regarded, the definition of involved agents will depend strongly on the use of the inquired FRP system. In fact the method concurs to neglect the agents, which don’t affect the durability of the inquired component. This is one of the advantages of the Method Nicolella compared with the other existing methods to estimate the service life of a building component. For example, it will be possible to disregard all agents related to the *rate of catch*, the wind, the snow and the UV exposure in the case of indoor applications such as plating of arch and vaults, where the agents related to the configuration became important.

The last phase of the method, schematically indicated in figure 2, features the evaluation of the modifying factor assumed by means of laboratory tests. In fact, taking into account the difficulties to define the correlation between accelerates laboratory tests and real exposure conditions and to appraise the validity of the extrapolation of results obtained for specimen to real structures, the method uses laboratory tests exclusively in order to estimate the relative incidence of the factors on the service life.

For this phase, the experimental tests illustrated in the first part of the paper, if opportunely ordered, could be used.

## 5 CONCLUSION

In spite of the numerous applications of FRP systems for the strengthening of existing structures, the Designers have not know enough about their durability. The available international guidelines introduce an approach that is often conservative compared to the proposed (Nicolella method). They do not supply a method to estimate the effective service life of the systems. Hence it is necessary to set standard tests for evaluating the reference life of these strengthening systems and to define methods, described in this paper, internationally recognized, for evaluating their ESL. The “Nicolella method” seems to be an instrument particularly adapted for this purpose (it is very easy to apply by computer systems), on condition that the *mid-normal* value of the service life of the system and the modifying factor’s values were correctly defined for the various typologies in mid-normal conditions.

## 6 ACKNOWLEDGEMENT

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# Condition Survey Data Warehouse: Analysing Data For Component Life Estimation

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**Summary**: **During the 1990s, the British Government required all providers of social housing to carry out surveys on their stock to ascertain its condition and maintenance requirements. As a result, a large body of data has been collected nationally. This study investigates the feasibility of analysing the data to estimate components' lives.**

**A data warehouse was built using data from one of the surveying firms involved in collecting the data (Building Performance Group). The dataset was processed to reduce ambiguity and standardise repair descriptions. Window and roof replacement data were extracted and the years to renew the components were analysed.**

**The data collected seems to be broadly in line with existing sources of durability information and indicates that collating data from different sources can be used to get a good estimation of the service lives of components and assemblies.**

**Keywords. Data warehouse, stock condition survey, durability, data mining.**

## 1 INTRODUCTION

During the 1990s, the British Government ordered all providers of social housing, such as local authorities and housing associations, to carry out surveys of their housing stock to assess funding requirements for repairs. These surveys were carried out directly by the local authority or by consultant surveying firms.

Building Performance Group have been undertaken many stock condition surveys over this time. The collected data represents the aggregated estimates of repair needs and times as perceived by a number of surveyors.

Compiling the data from these surveys provided a large data set that could be analysed to determine component lives. 6 surveys were chosen to test the hypothesis that condition survey data could be used to estimate component lives. These represented a number of different dwelling types such as flats, houses and high rise buildings.

## 2 METHOD

### 2.1 Data warehouse concept

A data warehouse is an interface for end–users to access specially prepared data to be used with decision support systems or executive information systems. It is typically a collection of data from a number of databases, categorised and denormalised into a flat–file database (table) which was used in this project to extract relevant information.

### 2.2 Software used

The software used for this prototype was a combination of Microsoft Access as the database management system (DBMS) and user interface. Minitab was used to output the statistical information. MS Access was chosen as it is in widespread use in the organisation. Minitab was used to get a feel for the type of reports required without writing routines in a dedicated report package such as Crystal Reports.

### 2.3 Databases

The survey data were stored in a number of formats, SQL Server (mdf), dBase (dbf) and MS Access (mdb) filetypes. Open Database Connectivity (ODBC) was used to link tables in order to process and clean the data and make new tables.

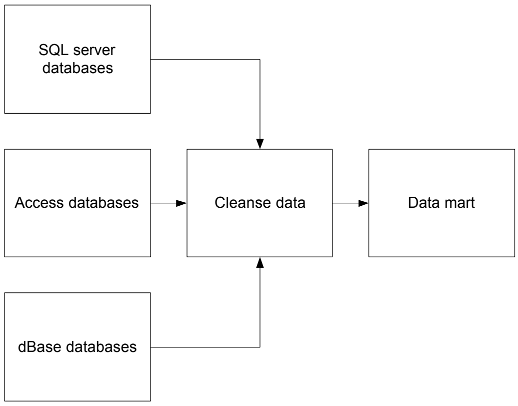


**Figure 1. Data warehouse architecture for condition survey data access**

### 2.4 Data warehouse

The warehouse database was used to query the survey databases to make new flat file tables containing the data fields described above. This table contained the original (unchanged) data as a 'look up' table to speed up processing and to ensure the integrity of the original tables was maintained.
The flat file table was copied for further data cleansing and processing. This proved to be a time consuming exercise. In particular, repair descriptions needed to be standardised. There were numerous ways of describing a particular repair and these were reduced to a generic component type repair field and a material field, for example, ‘repaint external door’ and ‘gloss paint to external door’ were altered to ‘decorate external door”.

### 2.5 Components analysed

Because of the amount of data cleansing in the repair description field, it was decided to use a subset of the components to test the system. External windows and pitched roof coverings were chosen as typical cases. This paper discusses the data and analysis on external windows.

### 2.6 Survey data

The data present in the database consists of a property identifier, post code, year-band of construction of the property, Building Cost Information Service (BCIS) code(BCIS, ), repairs and replacement requirements as determined by the surveyor, quantities and the year when the repair should be undertaken. The fields selected for inclusion in the data mart were: construction year band, repair year, survey year, post code, project identifier, property identifier, BCIS code, repair description, repair quantity and property type. The BCIS Standard Form of Cost Analysis for Building Projects is used throughout the UK to provide data which allows comparisons to be made between the cost of achieving various building functions in one project with that of achieving equivalent functions in other projects.
The quantity information has not been used for this exercise as it was felt that they could skew the results therefore each record refers to an actual individual observation by the surveyor but this data may be used for other analyses.

### 2.7 Data representation

Construction year band

The dates of construction were represented by the following year bands.

Pre 1920
1920-1935
1936-1945
1946-1960
1961-1980
Post 1981

In order to determine the years from construction to component renewal, it was necessary to represent the year bands by a specific year.

An adjusted construction year field was also added. This was to provide an estimate for prior renewal of components, for example: If the construction year was earlier than 1975 and windows are PVCu then adjusted construction year is 1985 or else the adjusted construction year is equal to construction year. The adjustments are described in the figure notes where appropriate.

### 2.8 Data processing

The data was collected over a number of surveys, in various locations in the United Kingdom over a period of 5 years. As a result, component and repair descriptions varied widely, particularly where the site surveyors recorded their results in free text rather than choosing from a predefined list. This was more marked where the data was collected using paper forms rather than electronically.

A repair record included a BCIS code. While this is an industry standard, it is a high level code and does not adequately describe repairs. The data was filtered by BCIS code and then the repair descriptions were standardised by using the search and replace capabilities of MS Access, inserting standard repair descriptions such as, 'Replace windows - PVCu' and finally by inspecting the remaining individual records and replacing the repair data with the standardised text. Where ambiguity still existed after this exercise, the record was discarded. From a total 250,000 records, 150,000 were considered usable for data mining. The discarded records were either ambiguous or appeared to be incorrect entries.

The database was then filtered for window data, using the BCIS coding system and filtering for 'window', 'wndw' and so on in the repair data field. This was resulted in 6590 records from 3014 properties where window replacement was unambiguously described.

## 3 RESULTS FOR WINDOWS

| Window Frame Material | Count | Percentage of Total (%) |
|---|---|---|
| Aluminium | 354 | 5.37 |
| hardwood | 108 | 1.64 |
| PVCu | 242 | 3.67 |
| Softwood | 5559 | 84.36 |
| Steel | 327 | 4.96 |
| N= | 6590 | |

**Table 1**: **Sample size and percentages of window types**

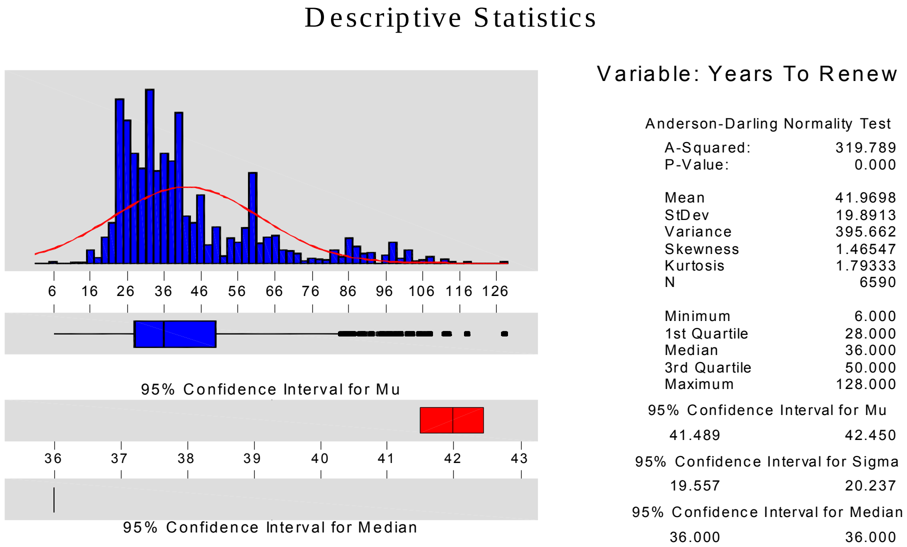


**Figure 2. All windows.**

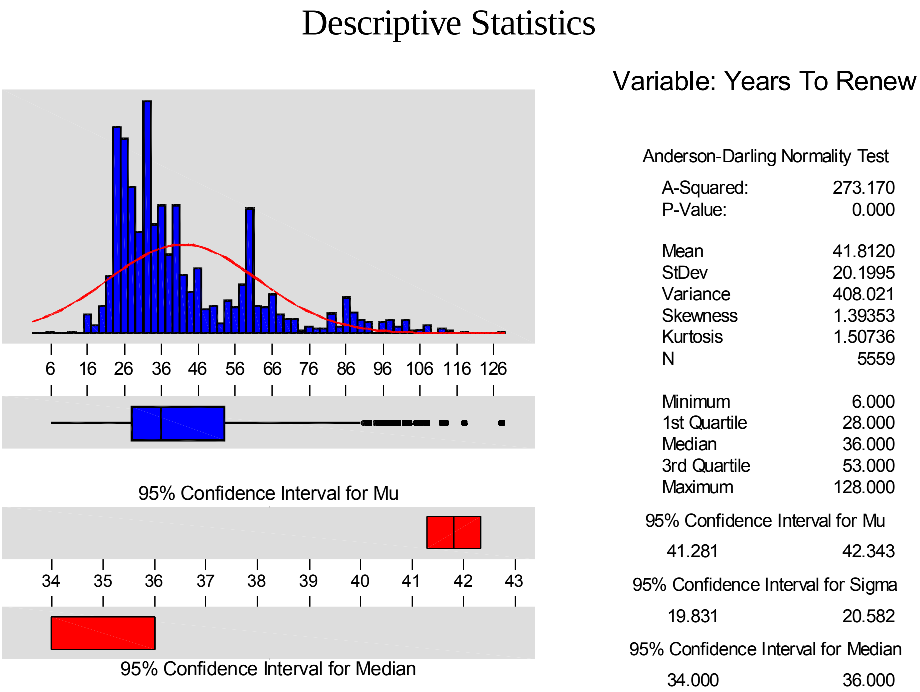


**Figure 3. All softwood windows**

The data seems to be skewed towards the left with a long tail towards the right. This is possibly due to the lack of information on component renewal after construction however there seem to be four distinct peaks which may indicate softwood windows of differing quality.

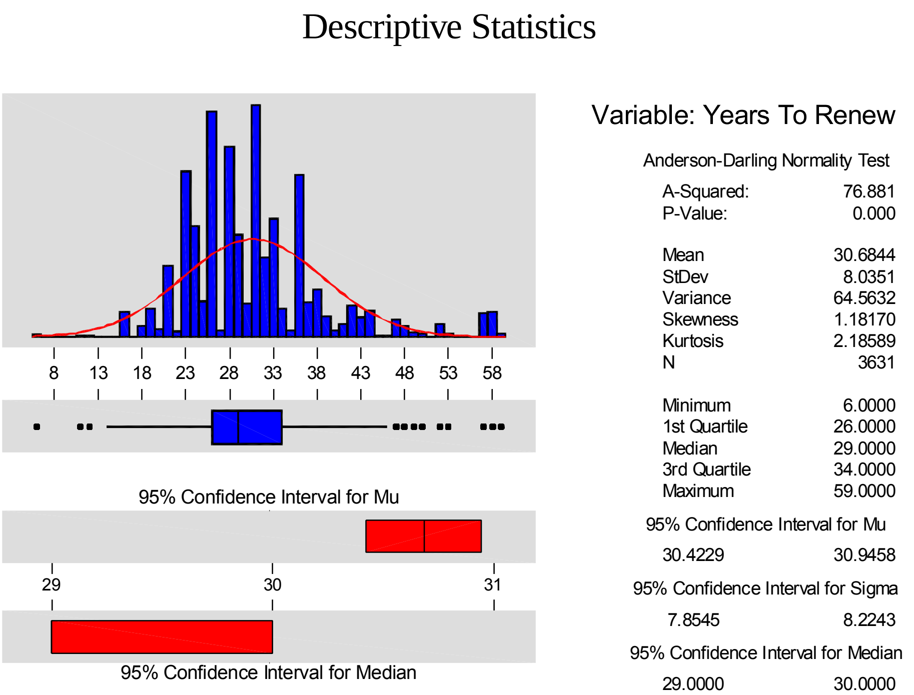


**Figure 4. Softwood windows. Date of construction >=1970**

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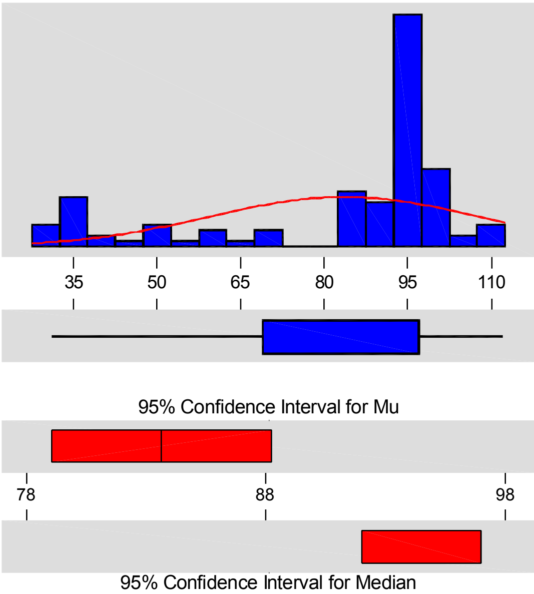
## Descriptive Statistics




Variable: Years To Renew

Anderson-Darling Normality Test

| | |
|---|---|
| A-Squared: | 11.447 |
| P-Value: | 0.000 |
| Mean | 83.6204 |
| StDev | 24.0061 |
| Variance | 576.294 |
| Skewness | -1.15945 |
| Kurtosis | -1.5E-01 |
| N | 108 |
| Minimum | 31.000 |
| 1st Quartile | 69.000 |
| Median | 97.000 |
| 3rd Quartile | 97.000 |
| Maximum | 112.000 |

95% Confidence Interval for Mu

79.041 88.200

95% Confidence Interval for Sigma

21.176 27.717

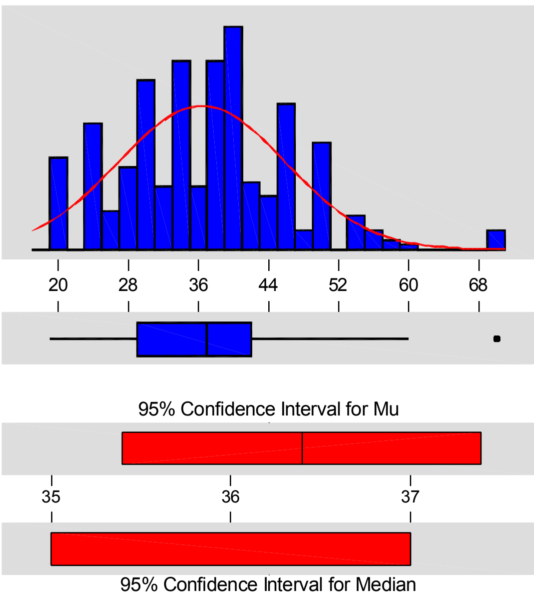
95% Confidence Interval for Median

92.000 97.000

**Figure 5. Hardwood windows**

While the data sample is relatively small, the mean is very high and may indicate that the windows were replaced at some time.

## Descriptive Statistics




Variable: Years To Renew

Anderson-Darling Normality Test

| | |
|---|---|
| A-Squared: | 1.446 |
| P-Value: | 0.001 |
| Mean | 36.3955 |
| StDev | 9.5574 |
| Variance | 91.3446 |
| Skewness | 0.487065 |
| Kurtosis | 0.622464 |
| N | 354 |
| Minimum | 19.0000 |
| 1st Quartile | 29.0000 |
| Median | 37.0000 |
| 3rd Quartile | 42.0000 |
| Maximum | 70.0000 |

95% Confidence Interval for Mu

35.3964 37.3945

95% Confidence Interval for Sigma

8.9014 10.3186

95% Confidence Interval for Median

35.0000 37.0000

**Figure 6. Aluminium windows.**

It was assumed that if the construction year <= 1960, then the windows were renewed in 1970.

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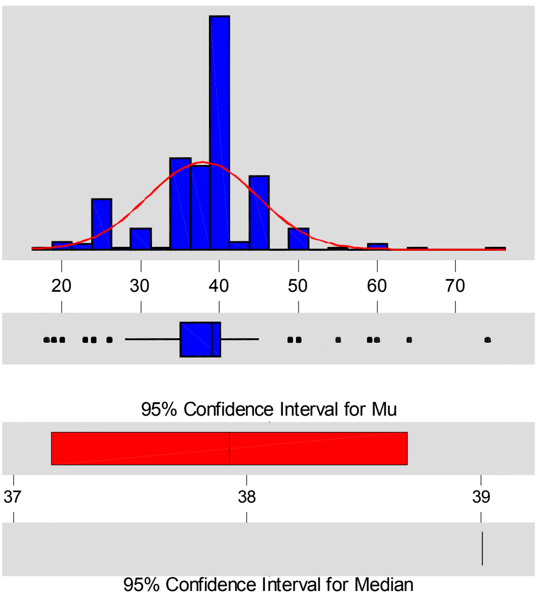
## Descriptive Statistics




Variable: Years To Renew

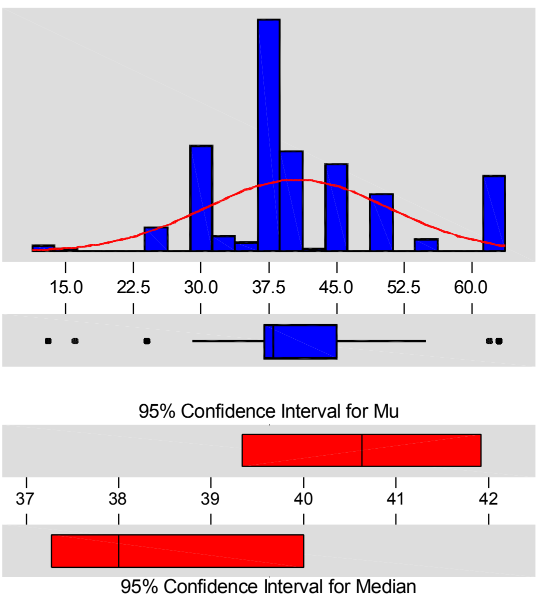
Anderson-Darling Normality Test

| | |
|---|---|
| A-Squared: | 14.027 |
| P-Value: | 0.000 |
| Mean | 37.9235 |
| StDev | 6.9836 |
| Variance | 48.7702 |
| Skewness | 0.105785 |
| Kurtosis | 3.40473 |
| N | 327 |
| Minimum | 18.0000 |
| 1st Quartile | 35.0000 |
| Median | 39.0000 |
| 3rd Quartile | 40.0000 |
| Maximum | 74.0000 |
| 95% Confidence Interval for Mu | |
| 37.1638 | 38.6833 |
| 95% Confidence Interval for Sigma | |
| 6.4862 | 7.5642 |
| 95% Confidence Interval for Median | |
| 39.0000 | 39.0000 |

**Figure 7. Steel windows.**

It was assumed that if the construction year <= 1955, then the windows were renewed in 1965.

## Descriptive Statistics




Variable: Years To Renew

Anderson-Darling Normality Test

| | |
|---|---|
| A-Squared: | 8.880 |
| P-Value: | 0.000 |
| Mean | 40.6281 |
| StDev | 10.2152 |
| Variance | 104.351 |
| Skewness | 0.649125 |
| Kurtosis | 0.548616 |
| N | 242 |
| Minimum | 13.0000 |
| 1st Quartile | 37.0000 |
| Median | 38.0000 |
| 3rd Quartile | 45.0000 |
| Maximum | 63.0000 |
| 95% Confidence Interval for Mu | |
| 39.3346 | 41.9216 |
| 95% Confidence Interval for Sigma | |
| 9.3790 | 11.2165 |
| 95% Confidence Interval for Median | |
| 37.2792 | 40.0000 |

**Figure 8. PVCu windows.**

It was assumed that if the construction year <= 1975, then the windows were renewed in 1985.

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## 4 DISCUSSION

The descriptive statistics graphs show the histogram and normal curve of the distribution of years to renew by quantity. The P-Value is low and indicates that the data is not normal throughout all types.

The component lives seem to be higher than indicated by Ahluwalia and Shackford (Ahluwalia and Shackford, 1993), who state the life expectancy of wood or aluminium casements to be 10-20 years and the HAPM Component Life Manual (HAPM Publications Ltd, 2000)(0-35 years) but seem to be broadly in line with the estimated service life of components (ESLC) as determined by the factoring method (BS ISO 15686-1:2000, 2000). While not investigated in this preliminary study, there is scope to investigate the effect of environmental conditions on components from the demographic and environmental information available for post–codes.

## 5 RECOMMENDATIONS

### 5.1 Classification system

The use of a common code to categorise component type and material is essential to ease data analysis. Cleaning the repair description data is too time consuming for a data warehouse and as stated previously, the BCIS code is not sufficiently 'low level' for this categorisation.

An alternative coding system such as the Uniclass classification system provides a suitable level for this application, for instance, softwood windows in external walls can be represented succinctly by the code G251:G321 which comprises the codes for External Wall (G251) and Windows (G321). It is recommended that this classification is added to the data capture software or inserted as part of post-survey data processing. A publicly available classification system also enables the data to be linked to other databases such as the HAPM component database for comparison against the insured life of similar components (HAPM Publications Ltd, 2000).

### 5.2 Repair and renewal history

Unfortunately the date of renewal of a component was not available to the survey teams. It may be that this information is held on the local authority or housing association databases and should be requested at the start of a survey. While this information is generally not needed for a condition survey, it would enable much more accurate results on component durability to be obtained.

### 5.3 Data sharing

The data for a large number of stock condition surveys exists nationally. All too often, once the data is processed and the results delivered to the client, the database is unused.

A national condition survey data repository (similar to the BCIS scheme for construction and maintenance costs) would ensure that the data is made available to interested parties. For confidentiality, identifying fields such as addresses, could be removed and the system could be made available to participants and subscribers.

## 6 CONCLUSIONS

Data about quality was not collected so comparisons of components of varying standards cannot be made.

The component lives seem to be higher than indicated (Ahluwalia and Shackford, 1993) but broadly in line with components lives estimated using the factoring method.

The time taken to clean the data vastly exceeded expectations. While it was thought that data cleaning would be a time–consuming exercise, it turned out to be an extremely onerous task. A large number of records were unusable because of missing or ambiguous data, 150,000 useful records were retrieved from the original 250,000 records for all repair and replacement types.

The data collected seems to be broadly in line with existing sources of durability information and indicates that collating data from different sources can be used to get a good estimation of the service lives of components and assemblies.

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# Effect Of Elevated Temperature On The Properties Of High-Strength Concrete Containing Cement Supplementary Materials

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**Summary**: **High-strength concrete is a material often used in the construction of high rise buildings. In the case of unexpected fire, the building concrete elements such as columns, slab and walls will be subjected to extreme temperatures. In order to assess the performance of high-rise reinforced concrete members it is important to understand the changes in the concrete properties due to extreme temperature exposure. Since the high-strength concrete produced may contain various binder materials in addition to cement, it is also becoming necessary to investigate the influence of the binder material type on the concrete properties under elevated temperature exposure. This paper summarizes and discusses the degradation of the strengths and stiffness of high-strength concrete in relation to the binder material type. The results showed that the binder material type has a significant influence on the performance of high-strength concrete particularly at temperatures below 800°C. The influence of the binder material type is significantly decreased at temperature of 1000°C. The strengths and stiffness of high-strength concrete are reduced with the increase in temperature without any threshold temperature level. The strengths are susceptible to the elevated temperatures compared to stiffness of concrete. High-strength concrete containing silica fume seems to be more sensitive to elevated temperature.**

**Keywords. High strength concrete, thermal conductivity**

## 1 INTRODUCTION

Fire-induced collapse of the World Trade Centre in New York, USA with heavy casualties has high-lightened the importance of the performance of construction materials at elevated temperatures. Engineering properties such as strength and stiffness and the thermal properties such as conductivity and expansion of these materials have to be understood by the designers when selecting the alternative materials. There is no doubt that high thermal conductivity and expansion of steel in addition to losses in strength and stiffness have contributed to the collapse of the towers in front of millions of viewers within a relatively short period after the terrorist attack. The obvious question, now being asked by many, is that what could have been the performance of these towers if concrete was used instead of steel? Is it reasonable to expect better performance with reduced number of casualties without collapse? It is no doubt that the owners of the high-rise buildings, the developers and designers of high-rise buildings wish to find answers to these questions. In search of the answers for these questions, it is necessary to understand the effects of extreme temperature exposure on the properties of concrete used in the construction of high rise buildings. The trend of utilizing high-strength concrete reduced the thickness of the columns as well as increased the attainable building heights.

Thermal conductivity is an important property of concrete since it controls the propagation of heat in a concrete element. The cover to steel reinforcement acts as the barrier and help to control the heat conduction through the steel reinforcement. Based on the research reported by Malhotra (1956) and Marshall (1972) the following conclusions can be drawn. The thermal conductivity of concrete is reduced with the loss of moisture during heating. The aggregate type used in a concrete mixture has significant influence on conductivity. Concrete with lower cement paste content as in the case of high-strength concrete can be expected to have a lower thermal conductivity than for lean concrete mixtures. Binder material type influences the thermal conductivity in so far as water release occurs at high temperature from the paste containing cement supplementary material such as blast-furnace slag.

Malhotra (1956), Zoldners (1960), Davis (1967), Abrams (1971), Faiyadh (1989), Khoury (1992), and Noumowe et. al. (1994) had reported the effects of high temperature exposure on the properties of concrete. Several mechanisms have been identified for the deterioration of concrete due to high temperatures. These include decomposition of the calcium hydroxide into lime and water, expansion of lime on re-hydration, destruction of gel structure, phase transformation in some types of aggregate, and development of micro-cracks due to thermal incompatibility between cement paste matrix and aggregate phase. High strength concrete compared to medium strength concrete is more brittle; contains less water; and the solid particles are more compact.

Hence, the effects of high temperature on high strength concrete will be different to those with the medium strength concrete. Sri Ravindrarajah (1998) studied the effect of temperature up to 800°C and method of cooling on the compressive and tensile strength of high-strength concrete.

The binder material used in high-strength concrete mixtures varies widely since the performance based specification for concrete allows the utilization of cement supplementary materials such as fly ash, blast furnace slag and silica fume in the concrete mixtures. Research into the properties of high-strength concrete in relation to the influence of mix compositions of concrete on its properties were reported elsewhere (Sri Ravindrarajah (1992), Sri Ravindrarajah *et. al.* (1993, 1994, 1995, 1998)). The main purpose of this study is to report the results of an experimental investigation into the effect of high-temperature exposure up to 1000°C on the properties of high-strength concrete as affected by the binder material type used.

## 2 EXPERIMENTAL DETAILS

### 2.1 Materials and mixture proportions

High strength concrete mixtures are generally characterized by low water to cement ratio and high binder material content. The use of high-water reducing admixture (superplasticiser) to achieve workable concrete at low water cement ratio resulted in producing concrete having its 28-day compressive strength well over 100MPa. In this investigation, four mixes were investigated having varying binder materials. Mixes 1 and 2 consisted 565 kg/m$^3$ of general purpose cement (Type GP), general blended cement (Type GB) having 62% blast furnace slag content. In Mix 3, 20% of the Type GP cement was replaced with low-calcium fly ash (115 kg/m$^3$). Mix 4 contained 565 kg/m$^3$ of general purpose cement and 40 kg/m$^3$ of condensed silica fume. For all four mixes, the water to binder ratio was kept at 0.30, by weight. Crushed basalt (specific gravity of 2.65) having the maximum aggregate size of 10mm and river sand (specific gravity of 2.62) were used as coarse and fine aggregate respectively. The aggregate to binder materials ratio was 3.0, by weight and the fine aggregate content was 35%, by weight. A constant dosage of superplasticiser (2% by weight of binder) was used in all concrete mixtures to obtain workable concrete mixtures.

The concrete mixtures were produced in a pan-type mixer. Freshly mixed concrete was tested for its density and used to cast a number of test specimens (standard cylinders and prisms) in steel moulds. A table vibrator was used to achieve full compaction for the moulded test specimens. The specimens used for compressive and tensile strengths were 100mm diameter by 200mm long cylinders. The concrete specimens used for dynamic modulus of elasticity testing had the dimension of 75mm by 75mm by 305mm prisms.

### 2.2 Heating equipment

A ventilated oven was used to heat the concrete specimens to a temperature up to 100°C. For the temperatures of 200°C and above, an electrically heated furnace designed for a maximum temperature of 1200°C was used. The furnace was heated by means of exposed heating elements laid on the refractory walls of the inside chamber, which was approximately 400 by 400 by 800mm in dimension. The test specimens were stacked with sufficient space between two adjacent specimens to obtain a uniform heating in each specimen. The test specimens were heated in batches due to limited capacity of the furnace. Extreme care was taken when handling the heated concrete specimens.

### 2.3 Experimental procedure

All concrete test specimens were demoulded after 24 hours and stored in water at 20°C. At the age of 28 days, the specimens were removed from water and dried for next two days in the laboratory environment, having the mean temperature and humidity of 20°C and 65% R.H., respectively. This step of pre-drying is necessary to minimize the explosion of concrete specimen when subjected to high temperature exposure in the furnace. The rate of heating was maintained at 200°C per hour. Once the required maximum furnace temperature was reached, this temperature was maintained until the specimens were removed. For all concrete specimens, the total duration in the furnace was 7 hours, irrespective of the maximum temperature. The maximum temperatures adopted in this study are 200°C, 400°C, 600°C, 800°C, and 1000°C. Therefore, the soaking period was reduced from 6 to 2 hours when the maximum temperature was increased from 200°C to 1000°C.

The test specimens were removed from the furnace after 7 hours and quenched in a water tank to provide the maximum thermal shock due to sudden cooling. A number of tests were conducted with the test specimens to determine the weight changes, ultrasonic pulse velocity, compressive strength, indirect tensile strength and dynamic modulus of elasticity. The split cylinders were used to study the variations in the colour of concrete due to heating to different maximum temperatures.

**Table 1. Properties of concrete mixtures at the age of 28 days**

| Mix No. | Binder materials | Cylinder strength (MPa) | Tensile strength (MPa) | Flexural strength (MPa) | Dynamic Modulus (GPa) |
|---|---|---|---|---|---|
| 1 | 100% GP Cement | 61.8 | 6.58 | 9.34 | 46.5 |
| 2 | 62% Slag + 38% Cement | 40.3 | 4.28 | 5.47 | 39.9 |
| 3 | 20% Fly ash + 80% Cement | 67.2 | 5.08 | 6.94 | 45.7 |
| 4 | 6.6% Silica fume + 93.4% Cement | 76.4 | 6.93 | 8.51 | 55.2 |

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## 3 RESULTS AND DISCUSSION

### 3.1 Properties of concrete mixes

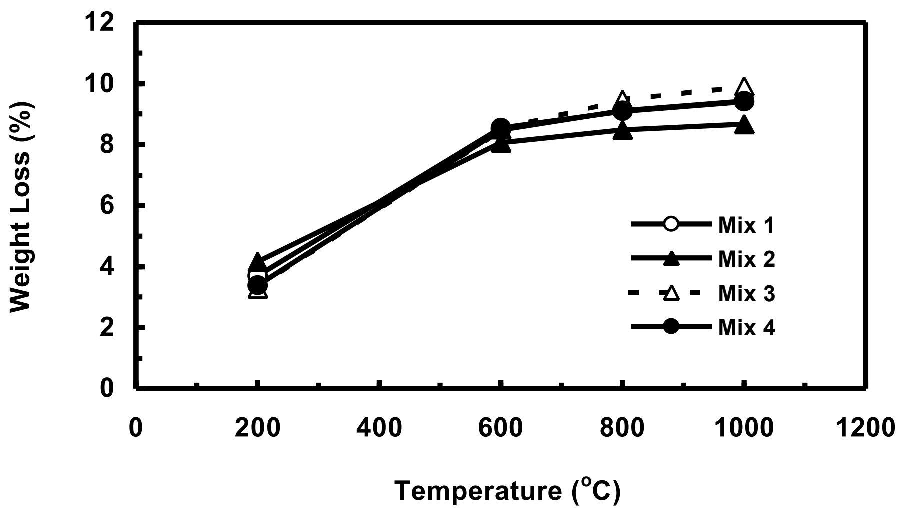
Table 1 summarises the properties of concrete mixtures at 28 days. The control concrete mixture (Mix 1) had the cylinder compressive strength of 61.8MPa. Mix 2 with blended cement, having 62% blast-furnace slag, showed considerably lower values for various strengths compared to the corresponding strengths for the control concrete mixture. The compressive, tensile and flexural strengths of Mix 2 are 65%, 65% and 57%, respectively of those for Mix 1. The dynamic modulus of elasticity for Mix 2 concrete was 39.9GPa compared to 46.5GPa for Mix 1. Hence, the elastic modulus was reduced by only 14% compared to 35% for the compressive strength. This reduced impact on elastic modulus for Mix 2 (as well as to Mixes 3 and 4) is probably due to the fact that the modulus is more sensitive to the aggregate stiffness than to the stiffness of the hardened paste. On the other hand, the strength of concrete is more sensitive to paste strength than the strength of the aggregate for normal weight concrete.

The results showed that the 28-day compressive strength of concrete, having 20% fly ash and 80% cement (Mix 3) was 8% more than that for Mix 1. However, the tensile and flexural strengths were 23% and 28% lower than the corresponding strengths for the control concrete. A marginal 2% reduction for the elastic modulus was observed. The addition of silica fume (Mix 4) increased in the strengths and stiffness of concrete. The compressive strength was increased by 24%, respectively, whereas the tensile strength was increased by 5%. However, surprisingly the flexural strength was decreased by 9%.

### 3.2 Weight changes in concrete on heating and cooling

Concrete is a porous material having discrete and interconnected pores of different sizes and shapes. The binder material type affects the pores size distribution. The use of cement supplementary materials such as fly ash, slag and silica fume will be result in achieving pore size refinement and grain size refinement. Water in the visible voids and large capillaries is termed as free water where as the water in the combined form in hardened cement paste is called combined water or water of hydration. Since the combined water is very stable it can only be removed at 1000$^{o}$C or above. In between these states, water exists in many locations such as in smaller capillary pores, gel pores and inter-layer spaces. The degree of firmness with which these water exists depends upon the pore size. Smaller the pore size is the greater the difficulty in removing the water.

**Figure 1: Effect of heating on weight loss in cylinders**



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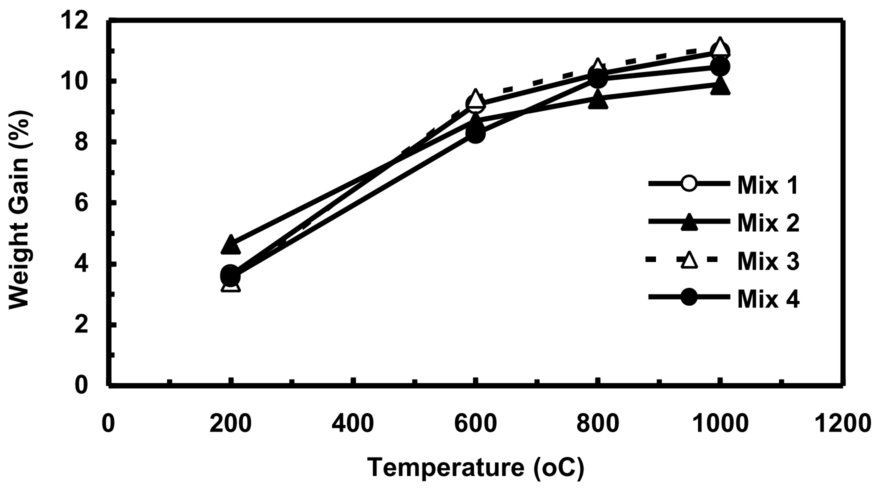


**Figure 2: Effect of heating on weight gain in cylinders**

When hardened concrete is heated gradually, the loss of weight appears to take place in the following three stages. In the first stage, called drying stage, evaporation of water from large capillaries and voids will take place. At the dehydration stage, which occurs between 100 and 600$^{o}$C, loss of non-evaporable water from gel pores and small capillary pores will take place. This stage is accompanied with substantial shrinkage of concrete. In the final stage, decomposition stage, several changes will occur in the concrete system. Some aggregate such as calcareous aggregate dissociates by releasing carbon dioxide at temperature over 800$^{o}$C. At the temperature above 1000$^{o}$C, the combined water form hardened paste will be released. This will cause the hardened paste to lose its cementing property and thus significantly reducing the hardened concrete properties. This process will be accompanied with significant shrinkage of cement paste as well as the loss in strength.

Figure 1 shows the weight loss in cylindrical specimens for all four concrete mixtures as the function of the maximum temperature. With the increase of exposed temperature, the concrete showed increased weight loss. However, the relationship between the weight loss and maximum temperature is non-linear. The binder material type caused a marginal influence to the relationship between the weight loss and the temperature. Fig. 2 shows the weight gained by the heated concrete as a function of the maximum exposed temperature. On sudden quenching in water, the heated concrete has gained more weight due to water absorption than the corresponding weight loss on heating. This could be due to the increased porosity of concrete related to internal cracking, in addition to any changes that may have occurred to the pore structure and properties of the hydrated paste on heating.

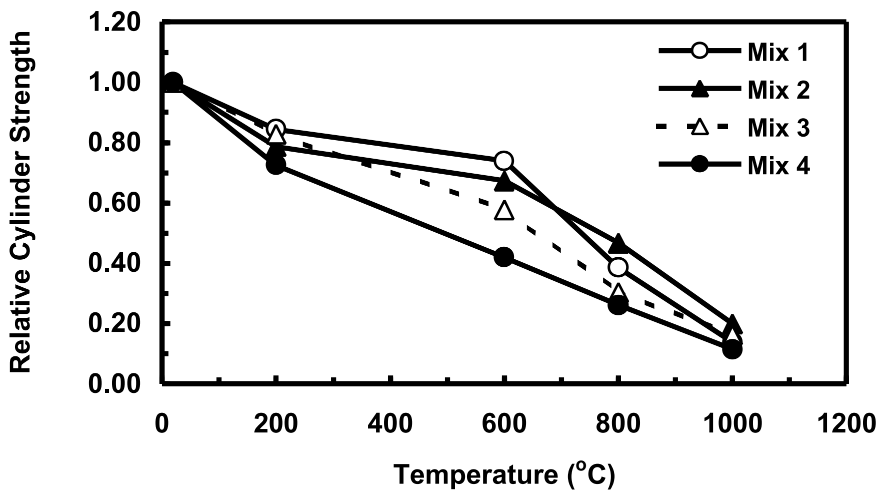


**Fig. 3: Residual compressive strength of concrete after high-temperature exposure**

**3.3 Effect of temperature on compressive strength**

Figure 3 shows the residual strength in relation to the original strength prior for all four concrete mixtures as a function of the maximum exposed temperature. The compressive strength of the high-strength concrete dropped significantly as the maximum temperature was increased. The residual strength varied from 73% to 84% of the corresponding initial strengths when the concrete was heated to the temperature was 200°C followed by sudden cooling in water. As the temperature was increased to 600°C, the residual strength was ranged from 42% to 74% of the unfired strength. Beyond 600°C, the strength continued to drop significantly and at 800°C and 1000°C, the residual strength ranged from 26% to 47% and from 11% to 20%, respectively. The results showed that the binder material type has a noticeable influence on the residual strength. The concrete mixture containing silica fume performed poorly compared to other binder materials. Although the silica fume addition increased the initial strength of concrete considerable compressive strength loss when exposed to high-temperatures. On heating to 1000°C, the compressive strength loss for the concrete with silica fume lost 89% of its initial strength.

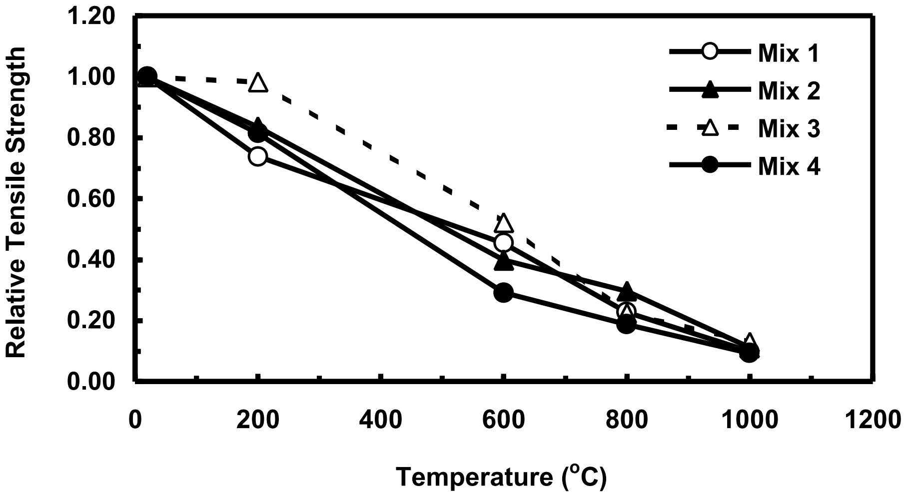


**Fig. 4: Residual tensile strength of concrete after high-temperature exposure**

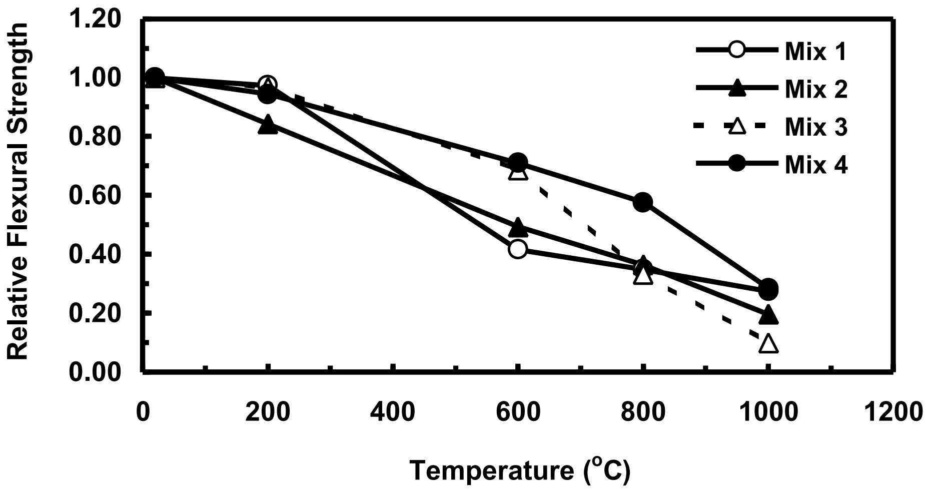


**Fig. 5: Residual flexural strength of concrete after high-temperature exposure**

### 3.4 Effect of temperature on tensile and flexural strengths

Figure 4 shows the residual tensile strength for high-strength concrete as a function of the maximum temperature. Similar to the compressive strength, the tensile strength showed significant losses with the increase in the exposed temperature. Once again, the binder material types played a noticeable role in influencing the strength losses. The results shown in Fig. 4 indicated that the Mix 4 having silica fume suffered significant loss in strength at the temperature of 600°C.The tensile strength for high strength concrete mixes was reduced by 87% to 91% when they were heated to 1000°C.

Figure 5 shows the effect of heating on the flexural strength for all four concrete mixes in relation to the corresponding strengths before heating. Similar to compressive and tensile strengths of concrete, the flexural strength decreased with temperature. Once again the binder material type had influenced the extent of strength loss.

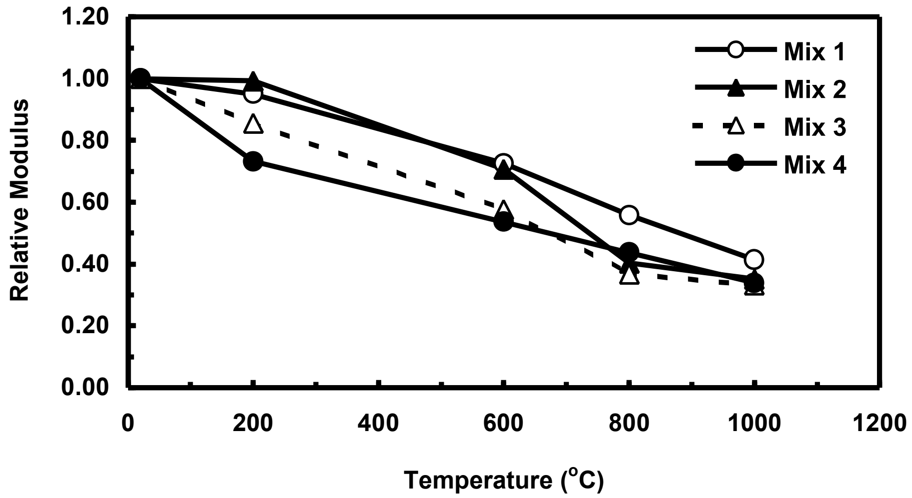


**Fig. 6: Residual modulus of elasticity of concrete after high-temperature exposure**

### 3.5 Effect of heating on modulus of elasticity

Figure 6 shows the residual modulus for all four concrete mixtures as a function of the exposed temperature. Similar to the strengths, elastic modulus was decreased with the increase in temperature and the binder material type influenced the extent of decrease. At 200°C, the reduction for the modulus ranged from 1 % to 5% and at 600°C it was ranged from 8% to 16%. Once the temperature was increased to 800°C, the reduction ranged from 13% to 20%. Further reduction was noted when the temperature was increased to 1000°C and the reduction in the modulus ranged from 29% to 40%, respectively. Therefore it appears from the results that the strengths of high-strength concrete are more sensitive to the elevated temperature than the elastic modulus.

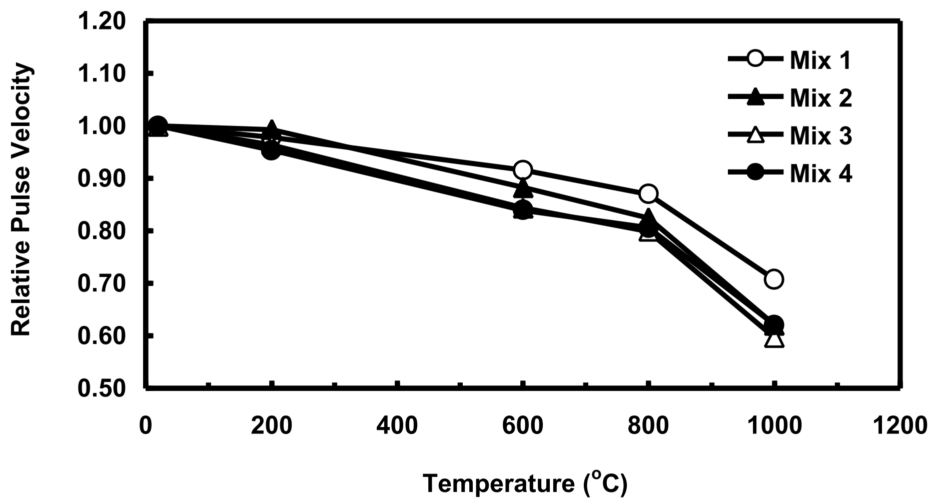


**Fig. 7: Residual ultrasonic pulse velocity of concrete after high-temperature exposure**

**Table 2. Effect of binder type on degradation in concrete properties with temperature**

| Binder materials | Relative Compressive strength | | | Relative Tensile strength | | | Relative Elastic Modulus | | |
|---|---|---|---|---|---|---|---|---|---|
| | 200 °C | 600 °C | 1000 °C | 200 °C | 600 °C | 1000 °C | 200 °C | 600 °C | 1000 °C |
| Cement | 84 | 74 | 14 | 74 | 44 | 10 | 95 | 73 | 44 |
| Cement + Fly ash | 83 | 58 | 20 | 98 | 55 | 13 | 85 | 58 | 33 |
| Cement + Silica fume | 73 | 42 | 11 | 82 | 29 | 9 | 73 | 53 | 34 |

### 3.6 Effect of binder material type on the concrete properties

Table 2 shows the influence of the binder material type on strengths and modulus of elasticity with respect to the corresponding values at the time of heating. The results have indicated that the concrete mixture containing silica fume loss considerable compressive strength compared to the control concrete or concrete with fly ash. The silica fume has contributed in improving the compressive and tensile strengths and elastic modulus of through the its pozzolanic reactivity it. However, concrete with silica fume performed poorly under the high temperature exposure.

The pore sizes are refined by this pozzolanic reactivity. However, water in the smaller pores will expand on heating and exert significant internal pressure within the system and thus result in the formation of internal micro-cracks. This has caused the ultrasonic pulse velocity to decrease noticeably compared to the control concrete as seen in Fig. 7. The initial pulse velocity ranged from 4.46 to 4.75 km/s was decreased to the range from 2.77 to 3.27 km/s when the temperature was increased to 1000$^{o}$C. The loss in pulse velocity is due to the combination of the effects of drying, internal cracking as well as to the changes in the microstructure of the paste on heating. These values are 60 to 70% of the initial values since the relationship between compressive strength and pulse velocity is exponential.

### 3.7 Effect of heating on the colour of concrete

Since the thermal conductivity reduces with the increase in temperature, water quenching of the heated concrete samples has not produced uniform cooling. The heat from the core of the test cylinders was unable to dissipate rapidly. Therefore, the core has sustained the elevated temperature for a longer period of time than the skin of the cylinders as observed from the colour differences across the section of the heated cylinders.

At the temperature of 200$^{o}$C, there was no apparent visual discoloration occurred in the concrete. The concrete specimens subjected to 600$^{o}$C and above suffered noticeable colour changes. The inner section of the concrete specimens at a distance of 15 to 20mm from the surface was dark grey-bluish in colour. The dark coloured area boundary was very distinct and showed no transition zone. Outside this area, concrete had maintained its colour. However, when the temperature was increased to 1000$^{o}$C, the inner colour was changed to bluish than dark grey.

## 4 CONCLUSIONS

Based on the results of this experimental investigation the following conclusions can be made:

1. High-strength concrete, independent of the binder material type used, experience weight loss and the relationship between the weight loss and maximum temperature was non-linear.
2. Compressive and tensile strengths showed noticeable losses (above 15%) even at the temperature of 200$^{o}$C, however the elastic modulus was dropped marginally by about 5%.
3. Concrete with silica fume suffered the most under increased exposure temperature below 800$^{o}$C.
4. High-strength concrete has shown 90% drop in its strengths once exposed to 1000$^{o}$C irrespective of the type of binder materials used.
5. Modulus of elasticity of high-strength concrete was less sensitive to the maximum temperature than either compressive or tensile strength.
6. Ultrasonic pulse velocity measurement can be used to estimate the temperature-related damage in concrete.

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# The Action Of The Lichen *Bacidia Sp* On White Cementitious Mortars In A Urban Environment

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**Summary: Some lichen species can damage the surfaces of calcitic rocks as dolomites and marbles, and also cement mortars and concrete by mechanical and chemical action. There are many lichen species known to colonize these substrates.**

**In the white mortar of a balcony with southern exposition in a urban environment in the center of the city of La Plata (Buenos Aires Province, Argentina) a lichen was collected and identified as *Bacidia sp*. This is very interesting, because in the searched literature there are no records of *Bacidia* species growing on mortars, but this species was originally found on calcareous rocks.**

**The original description mentions that this species has numerous fragments of the calcareous rock included in the thallus. SEM observations confirmed that this species penetrates the substrate. Macerate samples prepared by heating gently in hydrogen peroxide and the addition of potassium hydroxide were also observed. The aggregates are affected by the action of the lichen, and as a result superficial squamules were observed. There is also a low calcium content, as measured by EDAX microanalyses, confirming that this species has a detrimental action on cementitious materials.**

**Keywords: Lichens - Biodeterioration- White cement- *Bacidia*- Argentina.**

## 1 INTRODUCTION

Some lichen species can damage the surfaces of calcitic rocks as dolomites and marbles, and also cement mortars and concrete by mechanical and chemical action. There are many lichen species known to colonize these substrates (Nimis et al. 1987, Nimis et al, 1996).

As regards Buenos Aires Province, Argentina, some lichen species were found growing on “art –dèco” buildings (Traversa et al, 1999), and also on hydraulic structures (Rosato and Traversa, 2000). The most common species were *Caloplaca citrina* and *Lecanora albescens.* These species were studied with scanning electron microscopy (SEM), electron dispersive spectrometry microanalysis (EDS) and infra red spectrometry (IRS) (Traversa et al., 2000).

It was observed under SEM that *Caloplaca citrina* produces boring channels and *Lecanora albescens* leaves etching patterns on the surface. Both lichens produce a decrease in the calcium percentage of the mortar, as detected with EDS, whereas IRS analysis detected oxalic acid in both species.

In the white mortar of a balcony with southern exposition in a urban environment in the center of the city of La Plata (Buenos Aires Province, Argentina) a lichen was collected. The aim of this paper is to identify the species and to assess if it has a detrimental action on the substrate´s surface or not, in order to select an adecuate cleaning method.

It is necessary to remind that lichens are photosynthetic and therefore need the presence of light to grow, so they cannot penetrate a great depth in the substrate (only up to 2 mm.). Any damage they produce is only superficial, similar to other superficial attacks caused, for instance by weak acids together with the efect of freeze thawing cyles. Although it does not cause a problem to structural safety as a whole, this lichen is growing on a white ornamental cement cover and causes a grey, unaesthetic patch, so any deterioration induced by the lichen is relevant to select a technique for cleaning, specially when the building has an architectonical value.

## 2 MATERIALS AND METHODS

The lichen was found in a balcony with southern exposition in a urban environment in the center of the city of La Plata, in a private house built in 1958, growing along a fissure.This unifamiliar home belongs architectonically to the racionalist movement and the balcony is recovered by a white cementicious mortar (“marmolina”). This is a special ornamental mortar used as plaster made with white cement and marble bits and variable mica percentages. Its thickness is about 3-3 mm. and its is used to cover mortars prepared with normal cement.

The lichen sample was obtained by scraping it from the substrate and was studied with usual methods of identification. It was observed under stereoscopical and optical microscope. Measurements of thalli, apothecia, asci and ascospores were obtained. Keys were used for identification (Poelt, 1969, Ozenda and Clauzade, 1970 and Malme, 19).

Macerate samples prepared by heating gently in hydrogen peroxide and the addition of potassium hydroxide (Gehrmann, Krumbein and Petersen, 1988) were also observed with SEM (scanning electron microscope), and EDS (electron dispersive spectrometry) microanalysis were performed.

IRS analyses could not be performed, because the sample was too scarce.

## 3 RESULTS

### 3.1 Characteristics of the material

The white cement mortar used for the balcony is formed by one volume of white cement to three volumes of aggregate (small marble pieces and mica). It is most likely that a 10 % of hydrated lime has been added because that was the habitual technique used in La Plata during the 1950 decade.

The petrographic analysis of the aggregates shows little marble pieces, white cement, gypsum, mica and sub-rounded to sub-angular quartz clasts. Among the aggregates, it is striking that some magnetite crystals appear. The quartz and probably also the magnetite probably belong to the aggregate used in the base mortar and incorporated into the white cement during the execution. It has a specific weight of 2,35 at saturated density after drying the surface, and a water absorption rate in 24 hs. of 7,2 % .

### 3.2 Description of the lichen

Thallus crustose areolate, grey to dark gray, with a granulose surface, containing fragments of the substratum. Apothecia (structures that contain the spores), plane to convex, black disc. Spores hyaline, fusiform, with 3-5 septa, 21-30 x 3-3,5

The lichen is similar to *Bacidia subinnata* except for a great difference in the spore size. Nevertheless, in the searched bibliography there were no other descriptions similar to the specimen studied.

It is also interesting to remark that in the searched literature no records of *Bacidia* species growing on mortars could be found.

### 3.3 SEM observations

It can be seen that the thallus penetrates the substrate. When macerates were observed, it was noticed that the surface had a few boring channels and had squamules on the surface.

This is a remarkable feature, because this was not seen in other species. The most common damages are boring channels as in *Caloplaca citrina* and etching patterns as in *Lecanora albescens*, apothecial marks, perithecial cavities (Gerhmann, Krumbein and Petersen, 1988; Modenesi and Lajolo, 1988; Salvadori and Lazzarini, 1991), but in the available bibliography there are no description of squamules produced by the lichens.

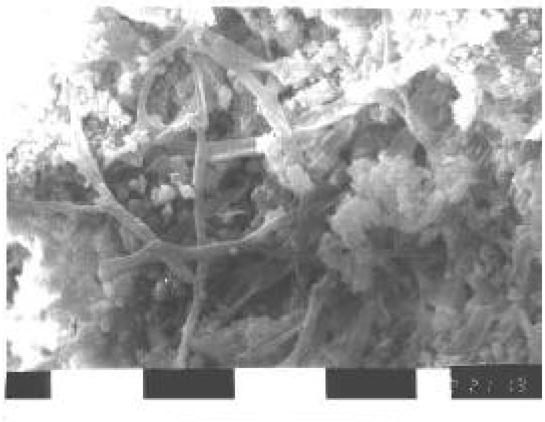

**Figure 1: Hyphae (fungal filaments) of *Bacidia* enclosing particles of the substratum.**

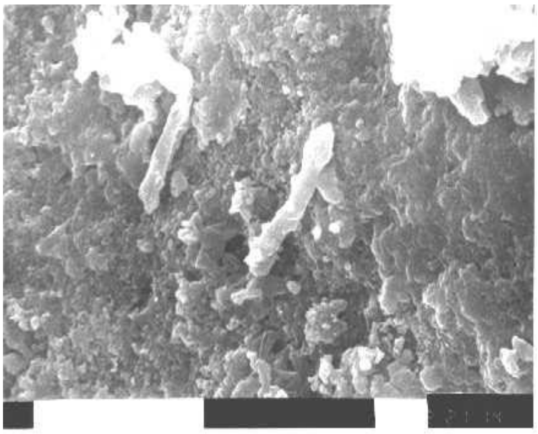

**Figure 2: Macerate showing the surface. Note the squamules.**

### 3.4 EDS microanalyses

The analysis detected a low calcium percentage (Table 1). There is a deterioration, because the material is easily disgregated when scraping, as it has been observed with other lichens. Nevertheless, the calcium loss is not as marked as in the cases of *Caloplaca citrina* or *Lecanora albescens* (Traversa et al., 2000). A normal cement, with 59% calcium, when attacked by

*Caloplaca citrina* had only 5 % calcium left. As for *Bacidia*, in the case being studied, the attacked cement has still a 35, 45 % calcium, although this may be attributed to the presence of marble in the aggregates, whereas the non attacked cement seems not to be so rich in calcium (the average is a 37,13 % and the maximun, 53,31 %).

It is also necessary to point out that white cements do not have any iron in the composition, and the percentage of iron detected is attributed to the presence of scarce magnetite crystals in the aggregates already mentioned.

Another difference with the typical white cement is that no sulphur or manganese were detected.

**Table 1- EDS results of the white cement mortar attacked by *Bacidia sp* and comparison to a typical white mortar**

| Samples | Elements | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Na | Mg | Al | Si | Cl | K | Ca | Fe | Mn | S |
| White cement mortar (mean values) | - | 9,41 | 10,44 | 39,25 | - | 0,745 | 37,13 | 2,63 | - | - |
| White cement mortar (lowest calcium data | - | 6,81 | 8,87 | 52,51 | - | 0,46 | 28,2 | 3,14 | - | - |
| White cement mortar (highest alcium data) | - | 11,22 | 7,34 | 25,37 | - | 0,19 | 53,31 | 2,58 | - | - |
| Attacked white cement mortar | 6,03 | 10,08 | 24,28 | 10,05 | 4,29 | 1,52 | 35,45 | 7,56 | - | - |
| Typical white cement (Blanks and Kennedy, 1955) | | 1,1 | 4,2 | 24,2 | | | 65,8 | 0,39 | 00,2 | 1,5 |

## 4 CONCLUSIONS

SEM observations confirmed that the lichen species *Bacidia* sp. penetrates the substrate, a white cement mortar with the addition of little marble bits and mica used as superficial plaster used to cover a normal cement plaster. The white mortar is affected by the action of the lichen, and as a result superficial squamules were observed. This is different from the attack of other lichens. The calcium decrease from a maximum of 53% to just 35 %, as measured by EDS microanalyses, seems not to be so important as the effect caused by other lichen species but the dissagregation of the material confirms that this species has a detrimental action on cementitious materials.

As a result of the lichen attack, there is a loss of structure and adherence between particles of the substrate, so in this case it is recommended that lichen removal should be done by washing with a solution of sodium hypochlorite and gentle brushing. Using hydrojet washing is discarded because the lichen has weakened the underlaying cement. The impact of the water jet would cause material loss and surface unevenness. Succesive washings can result in significant material loss, modifying the superficial texture and increasing the speed of colonization of this or other lichen species.

## 5 ACKNOWLEDGMENTS

To Mr. M. Sánchez, for the SEM observations and EDS analyses and geologist A. M. Ribot for the petrographic analysis. This work was partially funded by Conicet and CIC.

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# Conflicting Design Issues In Wood Frame Construction

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**Summary: In order for structures to provide adequate long-term performance, they must be designed and built to resist the imposed loads. These loads can be both structural and environmental in nature. This paper addresses performance issues related to design and materials selection as they relate to exposure from environmental elements. Of the environmental elements (e.g., moisture, fire, uv exposure), it is the contribution of the imposed moisture loads that usually result in the performance issues affecting durability. Buildings located in the urban-wildland interface (UWI) can also be exposed to the environmental load of wildfire. Testing has recently been conducted at the University of California Fire Research Laboratory whereby exterior building components and assemblies were exposed to simulated wildfire conditions. One of the results of these tests showed that construction details commonly used to protect a structure from moisture were often in conflict with those which would more effectively protect the same structure against the flame impingement and burning brand exposures typical for homes located in the UWI and subjected to wildfire. Examples of conflicting moisture-wildfire design issues include attic and crawlspace ventilation and roof overhangs. Traditional vents are vulnerable to flame and ember entry, but depending on the climate, are considered important from a moisture management perspective. Similarly, wide roof overhangs are considered a good design feature to protect cladding from rainfall, and can be good from a solar gain (energy conservation) perspective depending on location, but are a poor design feature from a flame impingement perspective. The objective of this paper is to present information on these conflicting design issues, and to explore how best to design structures located in the UWI.**

**Keywords: Design features, structures, urban-wildland interface (UWI), wildfire, rain**

## 1 BACKGROUND

Structures are designed and built to withstand certain imposed loads, as mandated by local and regional building codes. Traditionally we have been concerned about typical structural loadings (e.g., wind, earthquake and snow) that a structure could be exposed to. Depending on the likelihood of certain events that can have significant structural impact on the structure, regions can modify the basic building requirements to make the structure more resistant to a given imposed load.

For a number of reasons, improving the durability of structures has attracted more attention in recent years, and much of the attention has focused on protecting the materials on and within the building envelope against external exposures, the most common being moisture from both internal and external sources. Although construction details and design features related to moisture control are not always implemented, they are generally understood and have been well documented. Design features regarding energy performance of structures are also readily available, as well as those intended to make structures more "fire-safe". As is the case with moisture, fire-safe construction can address fires that start inside the structure, and those that result from exterior sources (wildfires).

Testing was recently conducted at the University of California Forest Products Laboratory (UCFPL) fire laboratory whereby exterior building components and assemblies were exposed to simulated wildfire conditions. One of the results of these tests showed that construction detailing and design features that were effective in protecting a structure from the infiltration of external moisture (usually in the form of rain) are very different that those that would be used to protect the same structure against flame and burning brand exposures typical for homes located in the urban-wildland interface (UWI).

Since there are many factors that could be considered when designing a structure, tradeoffs inevitably occur. One of the critical issues regarding the long-term performance of a structure is appropriate planning for the anticipated exposures, but in order for design professionals to make informed decisions, all of the exposure issues must be available. The objective of this paper is to

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present information on the conflicting design issues that exist between rain, wildfire, and, to some extent, energy conservation features, and to begin discussions on how best to design structures located in the UWI where wildfire (bushfire) poses a threat.

## 2 PERFORMANCE ISSUES: DESIGN OF COMPONENTS AND ASSEMBLIES

Similarities and differences were noted between design features that would be used to protect the exterior envelope from moisture/rain and those that would be used for wildfire. The similar design features for both moisture and wildfire design is the importance of proper detailing at the joints and penetrations. Obtaining adequate moisture and wildfire protection in the field of a given material or assembly (i.e., away from the edges) is the easiest to accomplish. Penetration of moisture and fire typically occurs at joints. This is why flashing details are so important when considering moisture management issues, and the same is true for fire penetration. The conflicting design issues deal with the "gross" design features and in the selection of materials. Examples of these gross design features include the width of the roof overhang, use of attic and crawlspace ventilation, and the spacing of deck boards in attached, spaced-board decks. These conflicting design issues will be addressed in the following sections.

### 2.1 Roof Overhang and Ventilation

The width of the roof overhang on a structure seems to be selected based more on the desired appearance of the structure rather than its ability to perform of a given function, even though there are clear benefits to narrow and wide overhangs, depending on the exposure. Publications dealing with performance and protection of building envelopes recommend wide overhangs to help deflect rain (Lstiburek 2000, Canadian Mortgage and Housing Corporation 1999), and narrow overhangs to provide protection against flame impingement and ember exposures common with wildfires (Moore 1981, Webster 1986, NFPA 1991). Germer (2001) reported on a procedure for determining roof overhang hased on the need for solar gain in the building, and is therefore related to energy conservation. In the procedure outlined by Germer, roof overhang is a function of the latitude where the structure is being built, and the vertical distance from the window sill to the roof overhang. In reports issued by the Canadian Mortgage and Housing Corporation (Rickens and Lovatt 1996), and Verrall (1966), walls with wider overhangs were associated with fewer water infiltration problems in the walls. Wide overhangs usually provide protection at wall penetrations in the field, but in cases where the base of the sheathing extends below the bottom of the sill plate, or the clearance between ground and sill isn't adequate, it can also minimize splashback, and the resultant damage to the bottom edge of the sheathing panel.

Roof overhang performance issues related to wildfire exposures are two-fold, one related to ventilation of attics, and the other related to flame impingement on the wall. Attic vents are frequently located on the underside of eaves, and have proven to be vulnerable to both entry of flames (flame impingement exposure) and glowing embers. Our research has shown that all forms of vents on the underside of the eaves (strip vents, frieze block, etc.), in both boxed and open-eave construction, are almost immediately penetrated under flame impingement exposures (Fig. 1). From this perspective, incorporation of metal or plastic strip vents with a noncombustible soffit material eliminates the material advantage in a fire safe design. The vulnerability of eave vents to fire has led to their elimination in some areas. The addition of through-roof vents on the roof surface can compensate for the loss of vent area at the eave, but it is questionable whether the attic area is being as effectively ventilated. Some have questioned the need for attic ventilation at all in hot humid climates (Lstiburek 1999, TenWolde and Rose 1999), and therefore in those climates moisture management and wildfire protection features may not be in conflict. In other climates, eliminating attic ventilation without incorporating other construction details that control the movement of moisture into the attic would not be wise. The use of screens in vents is intended to minimize or restrict the entry of embers into attic spaces, and is clearly more effective than vents without screens in this regard, but they do nothing to restrict penetration from a flame impingement exposure. Another feature that is being used in some regions is baffled vents (with the vents having either a horizontal or vertical orientation). The effectiveness of this design with regard to wildfire exposures has yet to be evaluated, but a similar design has been suggested for minimizing the entry of snow into attics during periods of high winds (Tobiasson 1994). Some local building codes in southern California suggest the use of (but do not require) baffled eave or soffit vents on new construction in the UWI. In these same localities, attic ventilation is not permitted on sides of homes fronting the wildland area (San Diego County 1997).

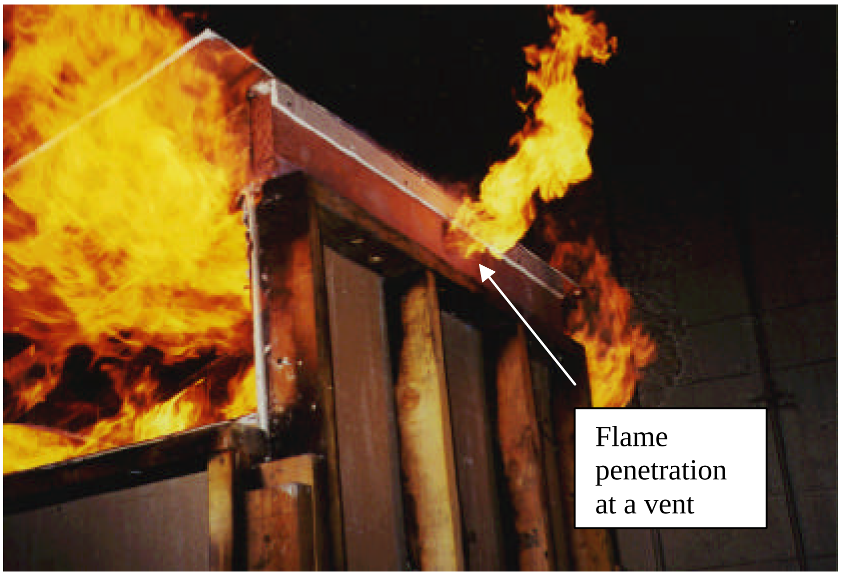



**Figure 1. This photograph shows flame penetration through a 2-inch diameter soffit vent located in the center of the wall, as viewed from what would be the interior of a structure. The flame impingement exposure simulates burning vegetation or exterior cladding.**

The second performance issue is related to flame impingement on the wall. The flame height on a wall is dependent on the entrainment of air into the flame plume (ASTM 1997). Because the flame is blocked on one side when it is against a wall, it will climb higher than one that is not in contact with a wall. Flames will climb higher yet at a corner. The flame plume will spread onto the surface of the eave (soffit) if an overhang exists due to the reduction of entrained air as the flame turns on the sloped surface (ASTM 1997). Results from research conducted at the UCFPL Fire Research Laboratory showed that flames would enter soffit vents located in an open eave (frieze block vent), and strip vents installed in boxed eaves almost immediately after a flame source was ignited at the base of the wall (Jennings 2000). A 600 kW propane diffusion burner was used, so the flame was able to immediately climb to the top of the wall. The strip vents were installed in 450 mm (18 inch) and 900 mm (36 inch) wide boxed eaves. With the 900 mm overhang the strip vent was located either 150 mm (6 inches) from the wall or 150 mm from the roof edge. If a combustible soffit material is used, wide overhangs can be more vulnerable to a flame impingement exposure even if vents are not used because more material is exposed. In the same study, failure in combustible soffit material occurred at joints in tongue & groove boards (Fig. 2) and at knots and core gaps in plywood soffits.


**Figure 2. Flame penetration occurred through a T&G joint in a soffit constructed with nominal 1-inch (25 mm) boards. This view is from the interior of the attic/soffit area.**

Crawlspace ventilation issues are similar to attic ventilation issues. Some level of crawlspace ventilation is required by code for moisture management, but again the effectiveness of ventilation has been questioned by some building scientists (Rose and TenWolde 1994). Crawlspace vents are often in close proximity to landscaping vegetation, which increase the chances of flame entry into the crawlspace should a wildfire reach the structure. Managing vegetation near a building (developing a 'defensible space') is always part of a firewise plan, and using such a plan will reduce the probability of wildfire reaching the structure and of fire penetration into attic and crawlspace vents. Other construction materials and details can be used to compensate for reduced venting that may be required by some codes. The use of a plastic ground cover in a crawlspace can reduce the need for ventilation, but not eliminate it (Quarles 1989), and the appropriate use of an air barrier can reduce the amount of moisture movement into the attic and building envelope. Slab on grade construction can also be used to avoid the crawlspace ventilation issue altogether.

Competing priorities create another conflicting design issue between aesthetics, "energy efficiency" and "firewise" constructions. As indicated by Wilson (2001), vegetation surrounding buildings can provide energy savings for a building, typically by improving the shading of a building. However, homeowners usually prefer vegetation surrounding homes, regardless of the other benefits and dangers.

### 2.2 Roof Overhang and Wall Interactions

The importance of a wide overhang in providing protection for the wall cladding by deflecting water from rain was discussed in the previous section. Our studies have shown that vertical and horizontal joints are the most vulnerable feature with regard to flame penetration into the building envelope. If flame penetration into the building envelope is going to occur in the cladding, it will typically occur at a joint (Fig 3). This implies that, at least for lapped siding, wider patterns would perform better under wildfire exposures because of the reduced number of joints that would be present on the wall. Edge thickness swell can be a problem with some wood-based composite siding materials, and from this perspective, is similar to the fire performance in that wider pattern would reduce the number of affected panels. Moisture related issues in siding are usually associated with biological degradation or dimensional stability (warp). Our experience has shown that warp-related defects are reduced when narrow patterns are used for wood and wood-based materials.

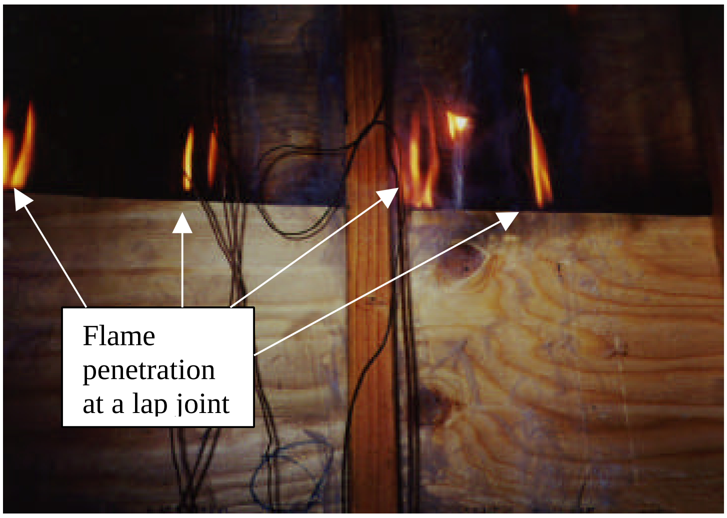
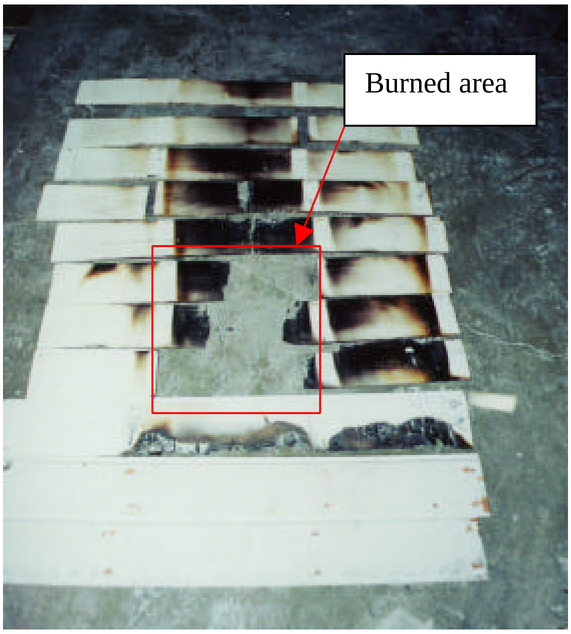



**Figure 3. This photograph shows flame penetration on the back side of one of the lap joints in panelized horizontal lapped siding. The flame source was on the front of the siding.**

In recent years use of a "rain-screen" design, whereby an air gap is included between the exterior cladding and the underlying sheathing has been discussed as a way of increasing the drying potential of the wall (Lstiburek and Carmody 1993; Quirouette and Rousseau 1998). Straube (1999) refers to this construction as a "screened-drained" wall assembly. This construction technique improves the drying potential of the wall, particularly if the wall is wetted by a leak in the building envelope. Preliminary tests conducted at this lab have shown that with this design damage to combustible siding materials increased, and because of this, the potential for burning through the building envelope also increases. Figures 4 and 5 show the backs of two wood-clad walls after being subjected to a flame impingement exposure for the same time period. The wall shown in Fig. 4 was attached to furring strips and the siding shown in Fig. 5 was attached directly to the building felt. The rain-screen wall suffered more damage as a result of charring and area burned. This result confirms those by Brannigan (1982) who discussed the increased fire susceptibility of a balloon-framed building envelope, whereby fire spreads throughout the cavity from the base of the wall to the roofline (and potentially into the attic). For rain-screen walls, the space between the cladding and sheathing provides a drainage plane, but also provides the confined space and oxygen for the flame to climb the back of the siding once the flame penetrates through the lap joint.




**Figures 4. The back of wood siding installed using a rain-screen technique after exposure to a flame impingement source. The damage to the siding was more severe than damage to similarly exposed siding attached flush against the building felt (see Fig. 5).**

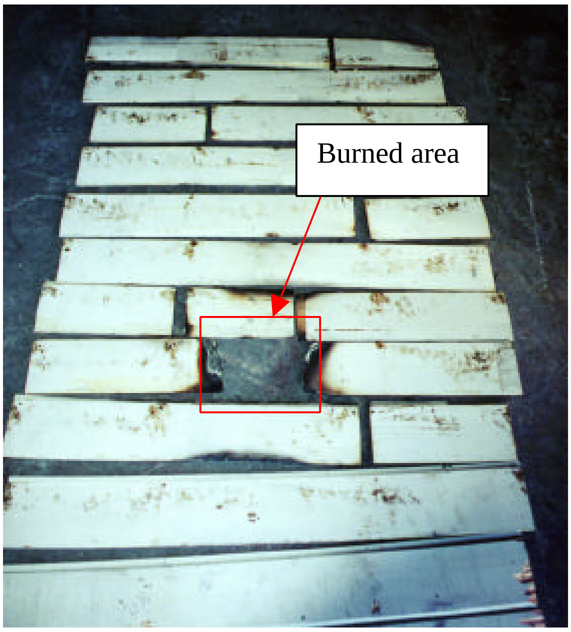



**Figure 5. The back of wood siding installed flush against building felt after exposure to a flame impingement source.**

### 2.3 Spaced-board Decks

Spaced-board decks obtain their best service life if the deck boards are spaced, and the between-boards gaps are maintained by clearing out the debris that can accumulate in the gaps. The air circulation that is facilitated by the gaps improves the drying potential of the deck boards, and the underlying support framing. However, these same gaps that allow for drainage and drying also provide radiant surfaces that enhance the burning, thereby increasing the rate at which the deck is degraded. These effects are shown in Figs. 6 and 7. These photographs show the increased fire hazard related to gapping the deck boards.

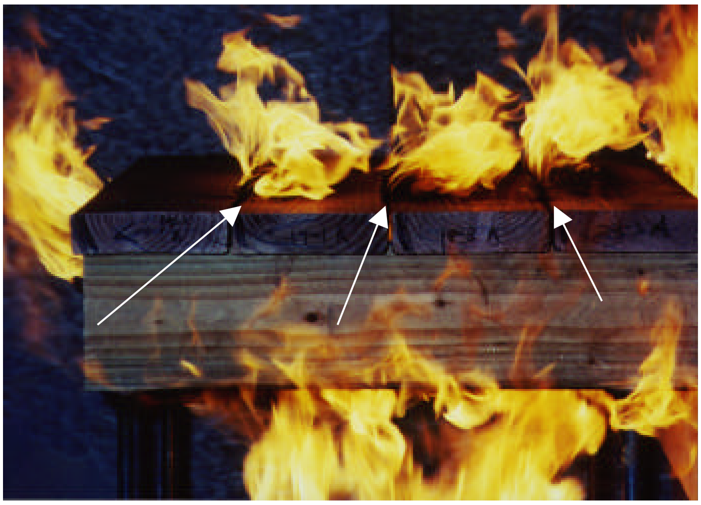


**Figure 6. Flame impingement extending upward through gaps between the deck boards in a deck subjected to a flame impingement exposure.**

**Figure 7. When deck boards are butted, flames cannot easily penetrate between board spaces. The rate of thermal degradation of the deck is minimized.**

Vertical screening and lattice systems are sometimes constructed around the perimeter of spaced-board decks in order to limit the entry of embers, and perhaps to discourage under-deck storage of materials. For large decks, the screen and lattice construction could limit ventilation, and therefore the drying potential, of wetted framing members, particularly in locations away from the perimeter where air circulation would be reduced.

## 2.4 Ventilated Membrane Decks

Wood-framed membrane decks (waterproof decks) have a solid surface and therefore would not have the conflict related to deck board spacing. Wood-frame membrane decks that are enclosed on the underside are usually vented, and therefore they would experience the same issues related to ventilation regarding ease of flame penetration. The vented membrane-deck shown in Fig. 8 is particularly vulnerable because of the growth of vegetation under the deck, and also because the building is located at the top of a ridge (not visible from the photograph). Flame entry into the joist cavities of these decks provides and easy access to the wall cavity, and therefore the structure.

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**Figure 8. The strip vents commonly used in woof-framed waterproof decks are subject to the same flame and ember entry problems as crawlspace and attic vents. In this case, vegetation growing under the deck increases the potential danger to the structure.**

## 3 SUMMARY

Basic design features that provide protection against rain and wildfire are often in conflict. For example, regarding exposure to rain, the performance of building components and assemblies can be enhanced if wide roof overhangs are used, if adequate ventilation of attics and crawlspaces is incorporated, and if the deck boards in spaced-board decks are gapped. If the structure design focused on maximizing performance assuming wildfire exposures, the opposite applies, and the roof overhang would be narrow, soffit and crawlspace venting would be eliminated, and deck boards in spaced-board decks would not be gapped at all. For homes built in the UWI it may be necessary to utilize the design features that are most effective against wildfire exposures, but this should not occur at the expense of the overall durability of the building. Other construction techniques and details must compensate for the anticipated change in performance. These techniques could include incorporation of air barriers to minimize the movement of moisture into the building envelope and use of vents that are designed to resist wildfire exposures but still allow sufficient movement of air to remove excess moisture. Elimination of the roof overhang would require greater attention to installation detailing at penetrations and may require a change to materials that are more dimensionally stable. Since many buildings are currently built with narrow overhangs, the need for more careful installation is understood. These changes may also mean that the time interval between normal maintenance tasks, such as painting and caulking, may have to be reduced.

Homes built in the UWI should be designed to perform well for all the anticipated exposures, and it is important to acknowledge and design for them. Trade-offs may have to be made, but this should not be at the expense of making the structure overly vulnerable to any given exposure.

## 4 ACKNOWLEDGEMENTS

I gratefully acknowledge the help of Chris Jennings in conducting tests during the time he was pursuing his Masters degree at the UCFPL. I also want to thank Professor Frank Beall, Director of the UCFPL, for allowing the flexibility in the wildfire research program to study some of the design issues reported in this paper.

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# Artificial Weathering Of Building Components - Experimental Results On External Walls

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**Summary: In this paper we present the results of a laboratory investigation conducted in collaboration with DISET-Politecnico di Milano on specimens of external walls subjected to artificial weathering.**

**Weathering was based on the repeated exposure of the specimens to the main weathering agents, such as rain, temperature, freeze/thaw cycles, humidity and sunlight. Each weathering cycle had a duration of about 6.5 hours.**

**The specimens consist of perforated brickwork coated with render and protected with paint. As protective layer an acrylic and a vinyl dispersion painting have been used, both with two different PVC pigment volume content (40% and 60% in volume).**

**The performance evaluation and the recording of changes in the samples properties have been made on the basis of visual inspection of the protective layer and of both non-destructive (specimens weight) and destructive tests (render compressive strength and porosity, render/brickwork bonding strength, water absorption, water vapour permeability and analysis of the microstructure of the paint/render system). The latter ones have been performed on cores taken from the wall specimens. The inspection of the paint and the non-destructive tests have been carried out every 25 weathering cycles during the whole exposure time, while the destructive tests have been performed only at the end of the exposure, after 50, 75, 150, and 325 weathering cycles. Characterization tests have also been performed on unweathered specimens.**

**The results of the tests show no significant variation of the mechanical properties and of the porosity of the samples, while a general rapid decrease as a function of the number of weathering cycles in the permeability to water and to water vapour of the paint/render system is observed. A stronger reduction of water absorption is observed for the specimens protected by the paints with a higher resin content, while surprisingly enough the permeability to water vapour shows a stronger reduction for the specimens protected by the paint with a lower resin content. The inspection of the paint and the analysis of the microstructure of the paint/render system showed that the degradation of the protective layer is much more rapid for the vinyl paint than for the acrylic one. After 150 weathering cycles the vinyl protective layer was indeed already characterized by the presence of a large number of lacerated bumps, crackles and detachments, while the acrylic one was practically undamaged.**

**Keywords: Artificial weathering, external wall, painting, durability.**

## 1 INTRODUCTION

The work presented in this paper is part of a wider research project jointly conducted by the DISET Department of the Polytechnic of Milan and the Laboratorio tecnico sperimentale of the University of applied sciences of southern Switzerland in Lugano. The issue addressed by the project is that of the durability of building components (Maggi *et al* 2001). The project aims, in particular, at developing an experimental method of service life prediction based on accelerated, field and in-use-conditions exposure of building components and on the evaluation of their performances. This type of method is considered by the international standard ISO 15686 “Buildings and constructed assets – service life planning” as one of the possible way for estimating service life of building components.

The paper reports on the results obtained from an investigation of the effects of short-term, accelerated exposure of wall specimens belonging to the class of traditional cavity walls to the action of artificial weathering agents. The investigation was carried out, on the one hand, in order to assess the decay rate and the extent of change of the building component’s critical properties and of those of its constituents. On the other hand, it was carried out in order to gather information on the mechanisms, which lead to a degradation of the building component's performances.

## 2 SPECIMENS AND EXPERIMENTAL

### 2.1 Specimens

The specimens, which represent only the external part of the cavity wall, consist of perforated brickwork coated with render and finished with a protective paint layer. Their surface has an area of about 0.2 $m^2$ and their weight is approximately 35 kg.

Fresh render mortar was prepared my mixing 1450 g of aggregates, 315 g of Portland cement type II/A-L 32.5, 135 g of limestone and 280 g of water (the quantities refer to 1 $dm^3$ of fresh mortar). The composition of the mortar has been defined by taking into account the directives on the preparation of render for external use issued by the Swiss Federal Laboratories for Materials Testing and Research (EMPA).

For the finishing of the specimens an acrylic and a vinyl dispersion paint have been used, both with 40% and 60% Pigment Volume Content (PVC). Each paint has been applied in three coats, each one having a thickness of about 30 µm.

Before starting their exposure to the weathering agents, the specimens were stored for at least 28 days in laboratory ambient conditions.

### 2.2 Testing program

Changes of the specimens’ properties due to artificial weathering have been monitored by means of visual inspection and by means of both non-destructive tests – weight measurement – and destructive tests – render compressive strength and porosity, render/brickwork bonding strength and water absorption capability, paint/render water vapour permeability. In addition, the microstructure of the paint/render system was examined by optical microscopy.

Visual inspection and weight measurements have been conducted every 25 weathering cycles during the whole exposure period, while the non-destructive tests have been conducted only at the end of the exposure, after 50, 75, 150 and 325 weathering cycles. For comparison, tests have been performed also on unweathered specimens.

## 3 WEATHERING APPARATUS AND WEATHERING CYCLE

The specimen were weathered in an environmental chamber, which allows to reproduce artificially the main weathering agents relevant to the investigated building component, that is rain, high and low temperatures, freeze/thaw cycles, humidity and sunlight. Inside the chamber, which has a volume of about 0.8 $m^3$, it is possible to regulate air temperature and relative air humidity between -30°C and +90°C and between 15% and 100%, respectively. For the simulation of sunlight, the chamber is equipped with a 6kW-xenon lamp, whose light spectrum is very similar to the spectrum of natural sunlight. The energy flux at the specimens’ surface is about 1500 $W/m^2$, while in a 10nm-wide spectral band centred at 365 nm (UV-range) the energy flux is 20 $W/m^2$. The chamber is also equipped with a spraying system fed with de-ionised water, which is used to simulate rain. Pictures of the environmental chamber are shown in Fig. 1.

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**Figure 1. Pictures showing the environmental chamber with two wall specimens.**

The control unit of the environmental chamber allows to programme every desired sequence of weathering agents and to set the duration of their action. The structure of the weathering cycle used for the present investigation is shown in Fig. 2. The cycle, which has been adjusted on the basis of the results obtained from exposure pre-tests (Teruzzi 2000), has a total duration of about 6.5 hours and consists of four main different phases:

Phase 1: during this phase, the specimens are homogeneously sprayed with water at 20°C for 60 minutes. Air inside the chamber is maintained at a temperature of 20°C and at a relative humidity of 95%.

Phase 2: air temperature is rapidly lowered from 20°C down to -20°C, where it is subsequently held constant for 90 minutes. During this phase, no control on air relative humidity is maintained. As shown measurements performed by means of sensors placed at different locations inside one of the wall specimens, at the end of this phase the temperature of the interface render-brickwork reaches about -10°C.

Phase 3: this phase reproduces conditions of higher temperature and humidity. Air inside the chamber is maintained during 60 minutes at a constant temperature of 55°C and at a constant relative humidity of 95%. This phase is preceded by a transition period during which air temperature is increased from -20°C up to the set value.

Phase 4: the specimens are exposed for 80 minutes to the artificial sunlight produced by the xenon lamp. A temperature of 30°C and a relative humidity of 40% characterise ambient conditions inside the chamber. At the end of this phase, the temperature of the interface render-brickwork reaches about 65°C.

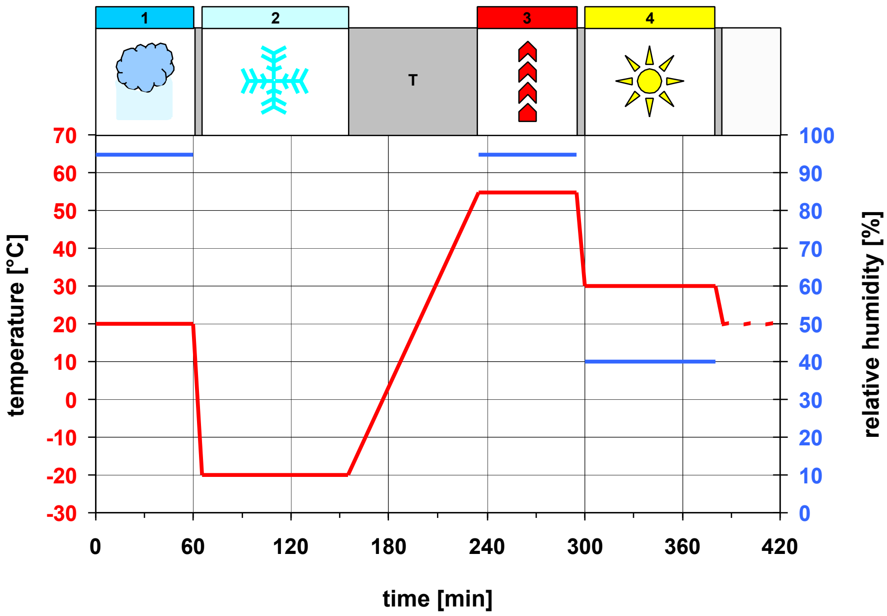


**Figure 2. Ambient conditions and agents sequence during each weathering cycle.**

## 4 RESULTS

In this section the results of the non-destructive tests performed on the wall specimens and the results of the destructive tests conducted on cores taken from the wall specimen are presented. Except for the graph shown in Fig. 6, all data points represent mean values calculated by taking the results obtained on the number of samples specified in each sub-section. For comparison, results obtained within the framework of a previous investigation conducted on uncoated specimens are also presented (Maggi *et al* 1999, Jornet 2000).

### 4.1 Paint's visual inspection

Changes in the appearance of the 40% PVC paints have been documented by means of pictures of the specimen's surface taken with a camera mounted on an optical microscope.

Figures 3, 4 and 5 show the surface of a specimen coated with 40% PVC vinyl paint after 75 weathering cycles, the surface of a specimen coated with the same type of paint after 325 cycles and the surface of a specimen coated with 40% PVC acrylic paint after 325 cycles, respectively.

After 75 weathering cycles, the specimen coated with the 40% PVC vinyl paint already displays signs of degradation. In fact, on the surface one can easily see many bumps, some of which are lacerated. After 325 cycles, the same paint shows a much higher degradation level. The surface is characterized by the widespread presence of lacerated bumps and detachments. In addition, the continuity of the protective paint layer is interrupted by a large number of crackles of different lengths and depths. In some cases, a white deposit around the largest crackles can be observed. This is due to the crystallization of salts that are dissolved inside the specimens during the spraying phase of the cycle (phase 1) and subsequently transported to the surface by water leaving the specimens during drying (phases 3 and 4). The appearance of the 40% PVC acrylic paint, after 325 cycles, apparently seems to be unaffected by the action of the weathering agents. In fact, no sign of degradation is pointed out by visual inspection. Basing on the results of visual inspection, the vinyl paint thus proves to degrade with a much higher rate than that of the acrylic paint.

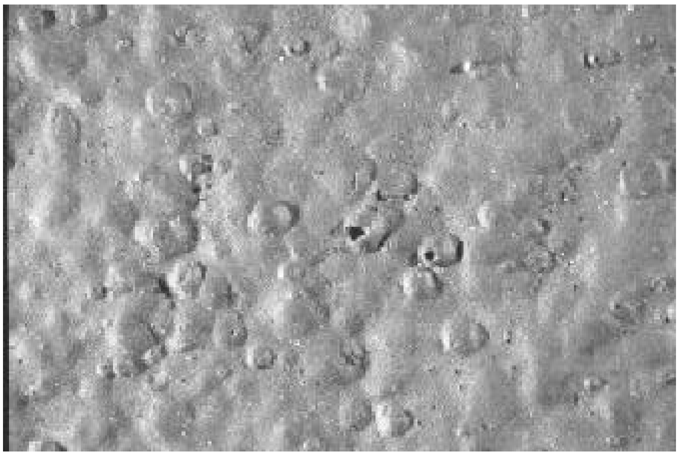

**Figure 3. Surface of the specimen coated with 40% PVC vinyl paint after 75 cycles.**

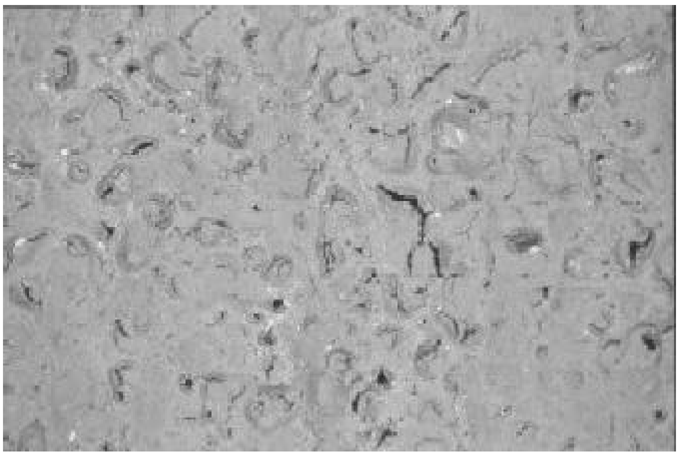

**Figure 4. Surface of the specimen coated with 40% PVC vinyl paint after 325 cycles.**

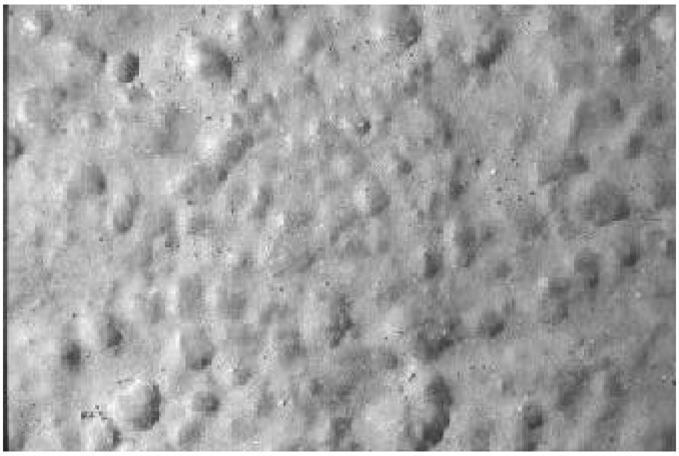

**Figure 5. Surface of the specimen coated with 40% PVC acrylic paint after 325 cycles.**

### 4.2 Weight

The specimen weight has been recorded after every series of 25 cycles in order to monitor the amount of residual water during weathering. It is important to notice that the lateral and rear sides of the specimens were watertight, so that during spraying water could enter the specimens only through the coated front side.

The variation of the specimen weight, expressed in percentage of the initial weight, is shown in Fig. 6 for all the different types of paint. For comparison, the results obtained for uncoated specimens are also displayed.

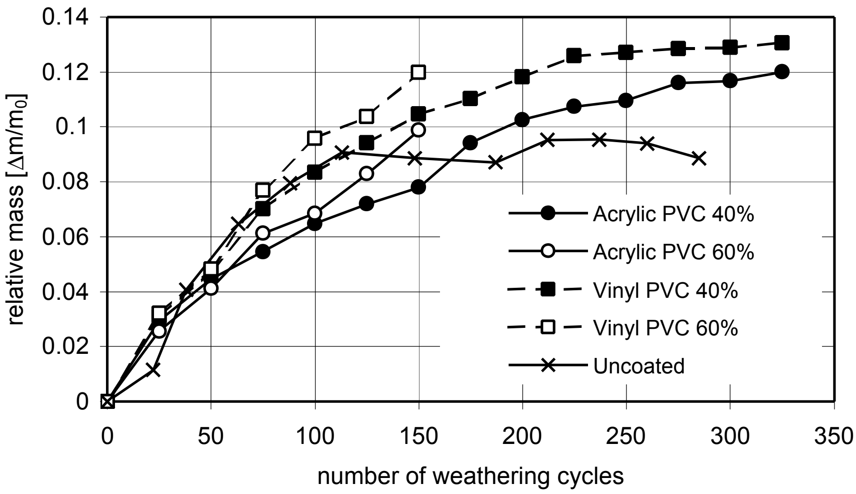


**Figure 6. Specimens' relative weight as a function of the number of weathering cycles.**

For all paints, a steady, asymptotic increase of the weight is observed. This fact is probably due to a positive balance between the amount of water that enter the specimens during the spraying phase and the amount of water that is released during the drying phases. After a sufficiently large number of cycles, the amount of water entering and that leaving the specimens balance, giving rise to the observed asymptotic behaviour.

Taking as a reference the data points at 150 weathering cycles, one clearly observes that the weight increases to a smaller extent for the acrylic paint than for the vinyl paint. Furthermore, as it becomes evident after about 50-75 cycles, for both types of paint, the larger increase is recorded for the highest pigment volume content PVC 60% (or, equivalently, the lowest resin content). As compared to the uncoated specimens, at steady state the relative weight of the coated specimens reaches higher values (12-13% for the coated specimens and 9% for the uncoated).

### 4.3 Render compressive strength and render/brickwork bonding strength

The render compressive strength and the render/brickwork bonding strength were determined according to the testing procedures described in the Swiss Standard SIA 162/1 and in the German Standard DIN 1040-2, respectively. The tests have been performed on three cores taken from each wall specimens. The results of the two tests are plotted in the graphs of Fig. 7.

With an overall mean value of (29.9 ± 4.5) N/mm$^2$, the render compressive strength does not show any significant dependence upon the number of cycles as well as upon the paint type. The results are practically identical with those obtained on cores taken from the uncoated specimens.

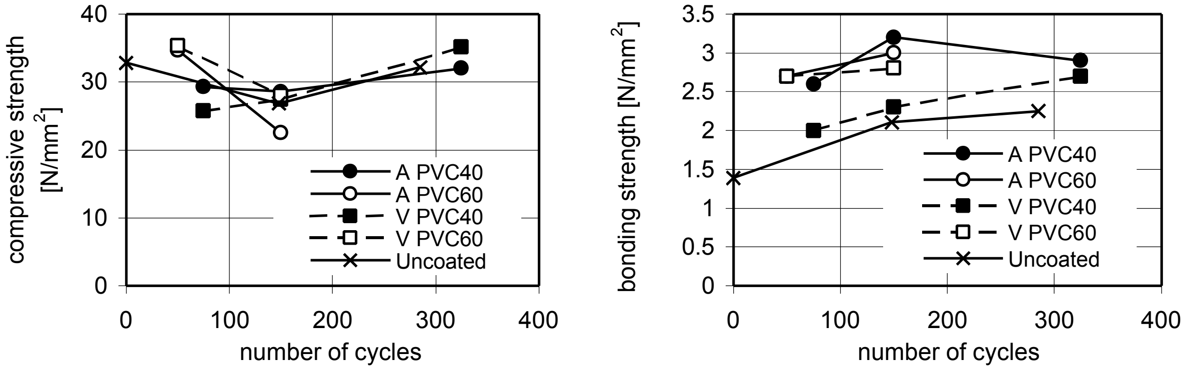


**Figure 7. Compressive and bonding of strengths as a function of the number of cycles.**

The bonding strength proves to be a slightly increasing function of the number of cycles for all types paint. The increase of the bonding strength is accompanied by a displacement of the rupture plane from the interface render/brickwork into the render. No significant dependence upon the paint type is observed. With values, which are comprised between 2.0 and 3.2 N/mm$^2$, the bonding strength obtained for the coated specimens is always larger than that obtained for the uncoated ones.

## 4.4 Porosity

The render's porosity parameters were measured according to the testing procedure described in the Swiss Standard SIA 162/1 (test no. 7). The test was performed on five cores for each wall specimen.

The results do not show any significant change of the total porosity and of the capillary porosity with increasing number of cycles. The total and the capillary porosity, as expressed in percentage by volume, show namely a constant value of about 23% and 13%, respectively, independently of the type of paint. In addition, the porosity parameters of the coated specimens are practically identical to those of the uncoated specimens.

## 4.5 Water absorption

The amount of water absorbed per capillarity by the paint/render system within a defined time interval has been determined on three cores for each wall specimen according the testing method given in the German Standard DIN 52 617. The results of the test are shown in Fig. 8 for t = 1 h (W1) and t = 24 h (W24), where t is the time elapsed since the beginning of the test. The results are expressed as the amount of water absorbed by a unit surface in the time t, divided by the square root of time (coefficient of water absorption W).

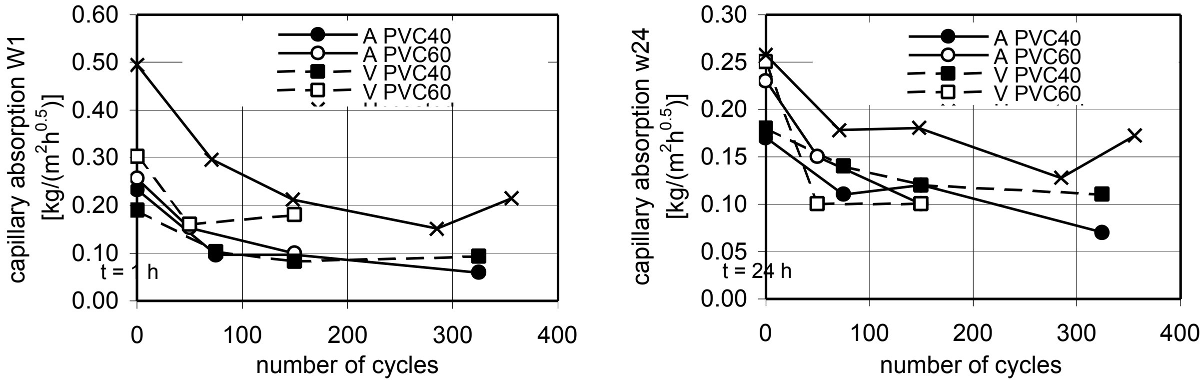


**Figure 8. Water absorption after 1 h (l) and 24 h (r) as a function of the number of cycles.**

For all types of paint a fast decrease of the coefficient of water absorption in the first 50-75 cycles is observed. This behaviour, which is observed also for the uncoated specimens, appears to be more important for the specimens coated with the higher PVC paints (PVC 60%). Afterwards, the coefficient of water absorption shows only a slight variation as the number of cycles increases.

With respect to the uncoated specimens, the coated ones are characterised by a coefficient of water absorption which, at t = 1 h, is smaller by about a factor of two. For t = 24 h differences in the absorption capability are less pronounced, but nevertheless the efficacy of the paint as a protective layer is proved. If one considers the coefficient of water absorption as an index of the paint/render system's protection level against water penetration, then the data displayed in Fig.8 clearly show, that the system coated with the lower PVC paint (PVC 40%) provides, for a not too large number of cycles, a higher level of protection than that coated with the higher PVC paint. This performance difference is less evident when the number of cycles is large. As far as the type of resin is concerned (acrylic vs. vinyl), no significant difference in the protection level is observed. It is noteworthy and quite surprising, that when the number of cycles is zero and t = 24 h, the system with a 60% PVC paint provides almost no protection.

In order to better appreciate the contribution to the protection level of the sole painting, water absorption tests have been repeated on samples from which the coating had been previously removed. Coherently with the results obtained from the monitoring of the specimen weight, one finds that the higher protection level is provided by the acrylic paint and that a lower PVC value helps to get better performances.

### 4.6 Water vapour permeability

Water vapour permeability and the corresponding resistance to water vapour diffusion were determined according to the testing method described in the German Standard DIN 52 615 on three cores for each wall specimen. The results are shown in Fig. 9.

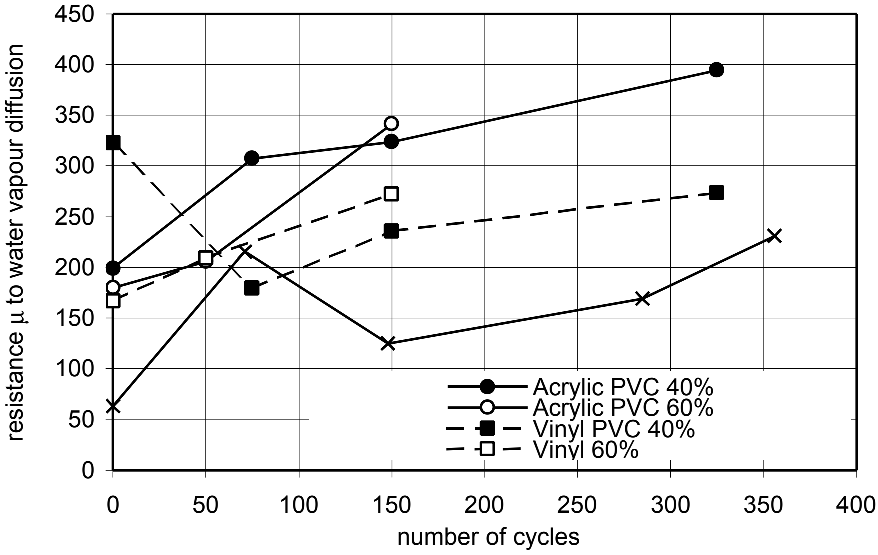


**Figure 9. Resistance to water vapour diffusion as a function of the number of cycles.**

The data points displayed in Fig. 9 clearly show that the presence of the paint results in an important increase of the resistance to water vapour diffusion. A general increase of resistance is also observed with increasing number of weathering cycles for all types of paint. Whereas for zero cycles no significant differences in the resistance are observed (neglecting the data point for the PVC 40% vinyl paint, which might possibly be incorrect), for a sufficiently large number of cycles the acrylic paint clearly shows a much higher resistance. In addition and surprisingly enough, for both the acrylic and the vinyl paints, the highest resistance values after 150 cycles are recorded for a pigment volume content of 60%, that is for the lowest resin content.

### 4.7 Microstructure

Examination, by means of optical microscopy, of thin sections prepared from samples of the paint/render system allow to point out microstructural transformations and alterations of the examined material owing to its exposure to the weathering agents. A thin section has been prepared for each type of paint and for each duration of exposure.

The pictures of Fig. 10 show the microstructure of the specimens coated with the 40% PVC vinyl paint after 0, 75, 150 and 325 weathering cycles. Whereas in the render no signs of alterations can be observed, the paint shows an increasing degradation level as the number of cycles increases. In accordance with the results of visual inspection, after 75 cycles bumps and bubbles start appearing. These are accompanied by detachments and ruptures after 150 cycles.

In the pictures of Fig. 11, the microscopic surface appearance of a specimen coated with the 40% PVC acrylic paint before exposure (zero cycles) and after 325 weathering cycles is shown. For this type of paint practically no sign of degradation can be observed.

The pictures of Fig. 12 show the microstructure of specimens coated with a PVC 60% acrylic paint and with a PVC 60% vinyl paint after 150 cycles. Although signs of degradation are evident in both pictures, the degradation level is significantly higher for the vinyl paint, which is characterized by ruptures and detachments.

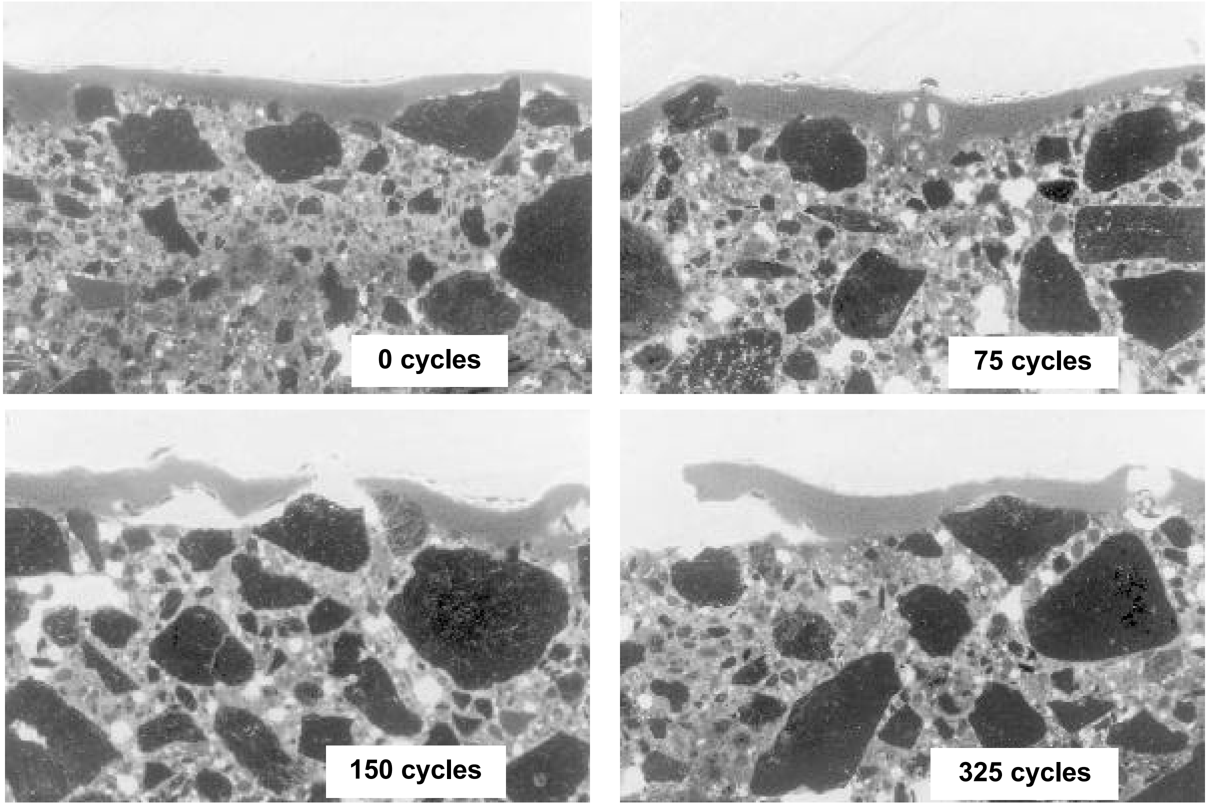


**Figure 10. Photomicrographies of the 40% PVC vinyl paint coating after 0, 75, 150 and 325 cycles. The largest dimension of the pictures corresponds to 2.7 mm.**

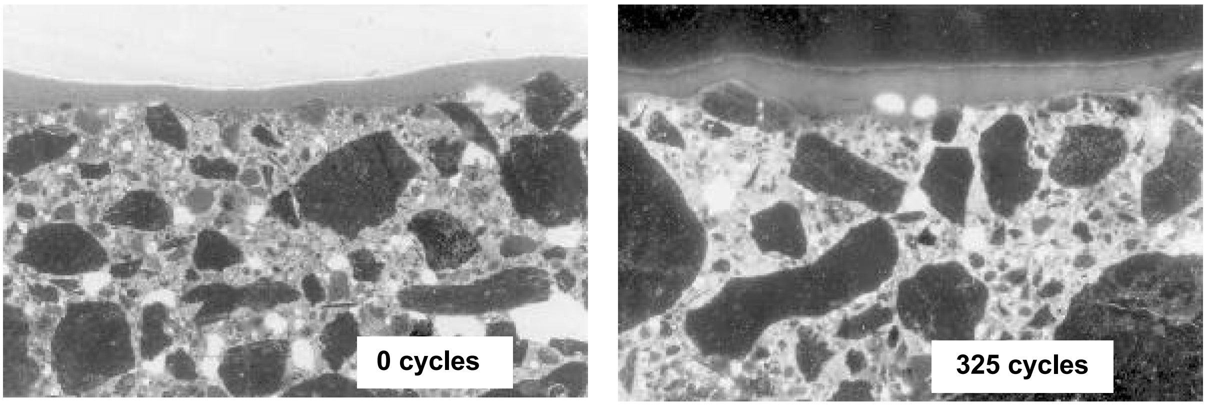


**Figure 11. Photomicrographies of a 40% PVC acrylic paint coating after 0 and 325 cycles. The largest dimension of the pictures corresponds to 2.7 mm.**

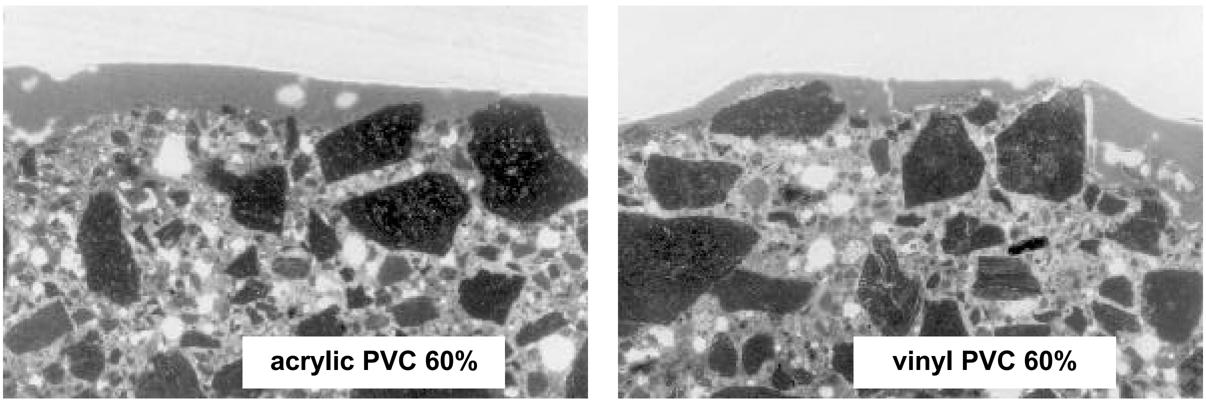


**Figure 12. Photomicrographies of a 60% PVC acrylic and vinyl paint coatings after 150 cycles. The largest dimension of the pictures corresponds to 2.7 mm.**

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By looking at the render, the quite large amount of air-entrained pores is standing out, which in all previous photomicrographies can be recognised as light structures. This feature has probably, strongly contributed to increase the resistance of the render to the many freezing and thawing cycles to which the specimens were exposed during weathering.

## 5 CONCLUSIONS

The results presented and discussed in the previous section proved that the weathering agents reproduced by the artificial weathering cycle have been able to induce significant degradation and/or alteration effects on the surface appearance of the wall specimens and on some of their physical and mechanical properties. Furthermore, for all the investigated properties, it was possible to determine the rate and the extent of change.

In particular, as shown by the results of visual inspection and of the microstructural analysis, the degradation of the vinyl paint develops with a higher rate than that of the acrylic paint. In addition, a lower pigment volume content helps to reduce the degradation rate.

The monitoring of the specimen weight and the determination of the water absorption capability of the paint/render system allowed pointing out the variable level of protection against water penetration provided by the different types of paint. In this regard, the acrylic paint proved to be better than the vinyl paint.

The experimental data presented in this paper are of course still not sufficient to predict the reference service life of the investigated building component. The data need to be compared and completed with those generated by field exposure, which is being conducted simultaneously in Lugano as well as in Milan, and with those which are going to be collected by in-use-conditions, accelerated exposure of the whole component. The results of field exposure will serve to check the realism of the effects resulting from accelerated exposure and for the re-scaling of the degradation rates, while those of in-use-conditions, accelerated exposure will be used to determine the minimum acceptable level of the component's critical properties.

## 6 ACKNOWLEDGMENTS

Thanks are due to the technical staff of the Laboratorio tecnico sperimentale for its invaluable work and to the colleagues of DISET in Milan for fruitful discussions.

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### 7.1 Standards

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DIN 52 617, *Bestimmung des Wasseraufnahmekoeffizienten von Baustoffen*, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin.

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# Reliability Of Timber Utility Poles Under Decay Attack

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**Summary: Reliability prediction is crucial to condition assessment, maintenance and management of utility poles, and, therefore, also important in ensuring the delivery of critical services (e.g., electricity) to customers. In this paper, methodology to predict lifetime reliability of timber poles is proposed, along with the consideration of in-ground decay attack. As the first stage, climate data including dry-bulb temperature, rainfall, and wind speed from Bureau of Meteorology (BoM) are used to acquire local environmental conditions around timber poles and subsequently find their probability density function. Secondly, the climate information is then applied to establish in-ground decay models, as well as extreme wind loading models. The eventual wind loading on the poles is not only dependent on the geometric size of the poles, but also relevant to the conductors attached to the poles and the distribution of neighbour poles as well. Thirdly, a limit state function is considered in terms of bending failure at the foot of timber poles, and the deteriorated failure resistance due to decay is indicated. Finally, decay-related conditional survival probability at a specified time and the lifetime survival probability are introduced on the hypothesis of extreme wind loading following a Poisson's process. Examples of utility poles involving 4 classes of timber materials are used in the prediction of time-dependent probability density functions of bending failure resistance, and the calculation of lifetime survival reliability. Results are expected to provide more efficient and accurate tools to risk management of utility poles.**

**Keywords: Timber, wood, decay, time-dependent failure resistance, lifetime reliability, utility poles**

## 1 INTRODUCTION

The utility pole industry in Australia spends about $40-50 million p.a. in maintenance and asset management because the cost and consequences of failure are very high. In NSW alone, with the replacement of one timber pole costing between $1000 and $10,000, depending on the size and attached conductors, the total replacement cost could reach $2 billion. However, the current approaches are to some extent based on qualitative estimation and engineers' experience, and hence may not be reliable and cost effective. Application of more efficient and accurate tools to predict the service life or reliability of poles will certainly contribute to cost reductions in both the maintenance and management of utility poles. A reduction in maintenance costs by 20% for power companies alone would save approximately $8-10 million every year.

The majority of utility poles in Australia are timber poles. As data from Department of Energy in NSW, treated and untreated timber poles have 40% and 52%, respectively, of the whole population of poles in NSW between 1994 and 1995. This gives even more importance to have an effective methodology in reliability prediction and management of timber poles.

As shown in Figure 1, a management system for timber poles may start from the establishment of an extensive database, ranging from BoM climate data, timber properties such as modulus of rupture, network distributions of utility poles, and other design parameters related to poles and conductors. Based on the database, performance modelling, evaluation and prediction are conducted in association with decay characterisation and reliability analysis. It is aimed to determine service-worthiness of poles and find potential measures that may be taken into maintenance to improve the service-worthiness. As known, a better decision is made, not only to satisfy safety requirements, but also to be able to control costs under acceptable budgets. To achieve high reliability at low cost is always the major objective in the management system. After the stage of decision-making, optimised strategies are then implemented through inspection, maintenance and replacement of timber poles. Results are monitored to confirm the efficiency and feasibility of the strategies, and information is finally fed back to modify the database for further analysis.

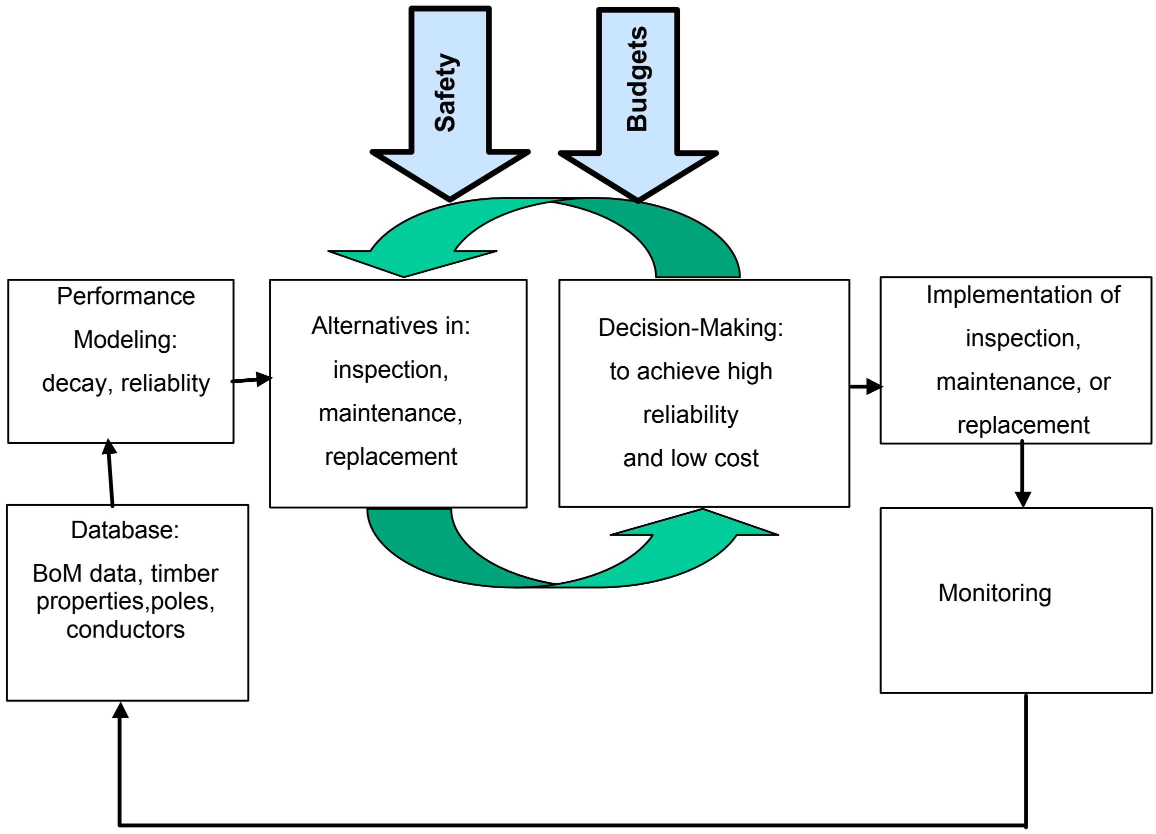


**Figure 1. A management system for utilities poles**

Undoubtedly, reliability analysis is the central issue in the risk-based management system. It is hence become a main subject discussed in this paper. To predict reliability of timber poles, three major factors need to be considered: loading, failure resistance and resistance deterioration.

Wind plays a significant role in the loading events applied on the poles and attached conductors, and it is considered as one of major factors causing poles to collapse. Wind load is dependent on the wind pressure, which is proportional to the wind speed as well as direction, both of which can be obtained from the data of BoM (Bureau of Meteorology). However, it is unnecessary to consider reliability based on all wind speed record. In current design procedures (ASCE, 1991), extreme wind speed is applied to estimate the nominal value that has a certain return period or occurrence frequency. The nominal value is then considered as a standard loading event in the reliability analysis. Therefore, understanding of distribution of the extreme wind speed is the first step to successfully predict the reliability of poles. It should be mentioned that the wind pressure is not only applied on the poles, but also applied on the conductors (cables) attached on the poles. The wind loading on the conductors along with the weight of conductors can produce a significant amount of concentrated force onto crossarms, and therefore, on the utility poles. The configuration of attached conductors and distribution of neighbour poles have also considerable effects on the wind-related loading. To some extent, loading events should be considered in a certain range of electricity distribution networks.

It is understood that timber materials are vulnerable to the attack of decay fungi and termites. Degradation of timber poles at the ground level has been one of the major concerns in the design and maintenance. As a direct consequence of decay attacks, the failure resistance constantly declines with service time. The rate of decay has been found to be mainly dependent on geographical location, timber species, preservative treatments, above- and in-ground climate index (Leicester et al., 2000; 2001). The climate index is proportional to the functions of yearly average temperature and yearly rainfall, both of which can be calculated from BoM data. Although calibration of decay models is still on the way in our Lab, it will not effect discussion about the methodology used to predict reliability of poles. However, the improvement of decay models can certainly increase the confidence of predicted results of reliability.

One of pivotal steps in the structural reliability analysis is to establish limit state functions, which are directly related to the loading events and failure resistance. The failure resistance is normally a random variable satisfying a probability density function such as normal and lognormal distribution, which can be determined from material property tests. Considering extreme wind events and related nominal values with certain return period, the survival or failure probability at a specified time – conditional survival/failure probability can be calculated by the use of Monte-Carlo simulation without difficulty. However, it is different from the lifetime survival/failure probability, which should account for the probability covering service life period, such as 10, 50 or 100 years for timber poles. When the occurrence of extreme wind events meets a Poisson's process in the time series, the lifetime survival/failure probability can be derived in terms of the conditional survival/failure probability and occurrence frequency of loading events. This is one of the most important parameters in the optimisation of risk-based management.

In the following, as examples of the methodology to analyse reliability of timber poles, lifetime survival probability of timber poles at four locations - Alice Spring, Darwin, Perth and Sydney of Australia, will be predicted, on the basis of local BoM data and timber decay models proposed in the paper.

## 2 RANDOM PARAMETERS IN RELIABILITY ANALYSIS

Temperature, rainfall, wind speed and direction are the main environmental conditions that have direct influence on the decay of timber poles and the loading on electricity transmission systems, and consequently on the structural limit state function and reliability analysis. Yearly rainfall and average temperature were suggested to be main factors in the modelling of decay of timber materials (Leicester et al., 2000; 2001), it is therefore important to understand the probability distribution of these two random climate elements. Also, the use of extreme value distribution of loading is often an effective method in the structural reliability analysis. It is imperative to calculate extreme wind speed distribution before extreme wind loads with a mean return period can be determined.

### 2.1 Yearly Average Temperature and Rainfall

From the data provided by BoM, the yearly average temperature in Alice Spring, Darwin, Perth and Sydney are calculated. The cumulative distribution functions at 4 locations are shown in Figure 2, fitted with normal distribution function which *m* (mean) and *s* (standard deviation) are given in Table 1.

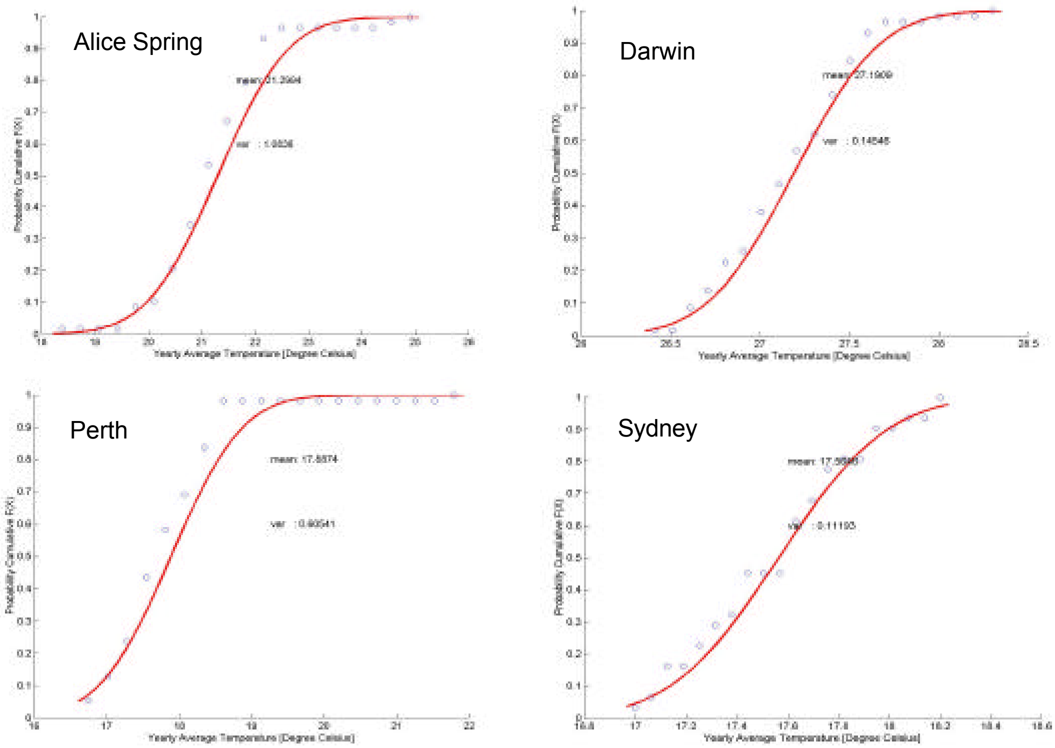


**Figure 2. Cumulative distribution function of yearly average temperature, fitted with normal distribution function.**

**Table 1. Parameters of the normal distribution function fitted to the yearly average temperature (°C) at four locations.**

| | Alice Spring | Darwin | Perth | Sydney |
|---|---|---|---|---|
| *m* | 21.30 | 27.19 | 17.89 | 17.56 |
| *s* | 1.041 | 0.3814 | 0.7781 | 0.3346 |

Yearly rainfall in Alice Spring, Darwin, Perth and Sydney can also be calculated from the BoM data. The cumulative distribution functions are depicted in Figure 3, and fitted with Weibull distribution function. Function parameters, *a* and *b*, are given in Table 2.

**Table 2. Parameters of the Weibull distribution function fitted to the yearly rainfall (mm)**

| | Alice Spring | Darwin | Perth | Sydney |
|---|---|---|---|---|
| a | $2.2189 \times 10^{-5}$ | $1.1123 \times 10^{-17}$ | $3.9899 \times 10^{-10}$ | $3.2158 \times 10^{-12}$ |
| b | 1.8537 | 5.1572 | 3.2264 | 3.7202 |

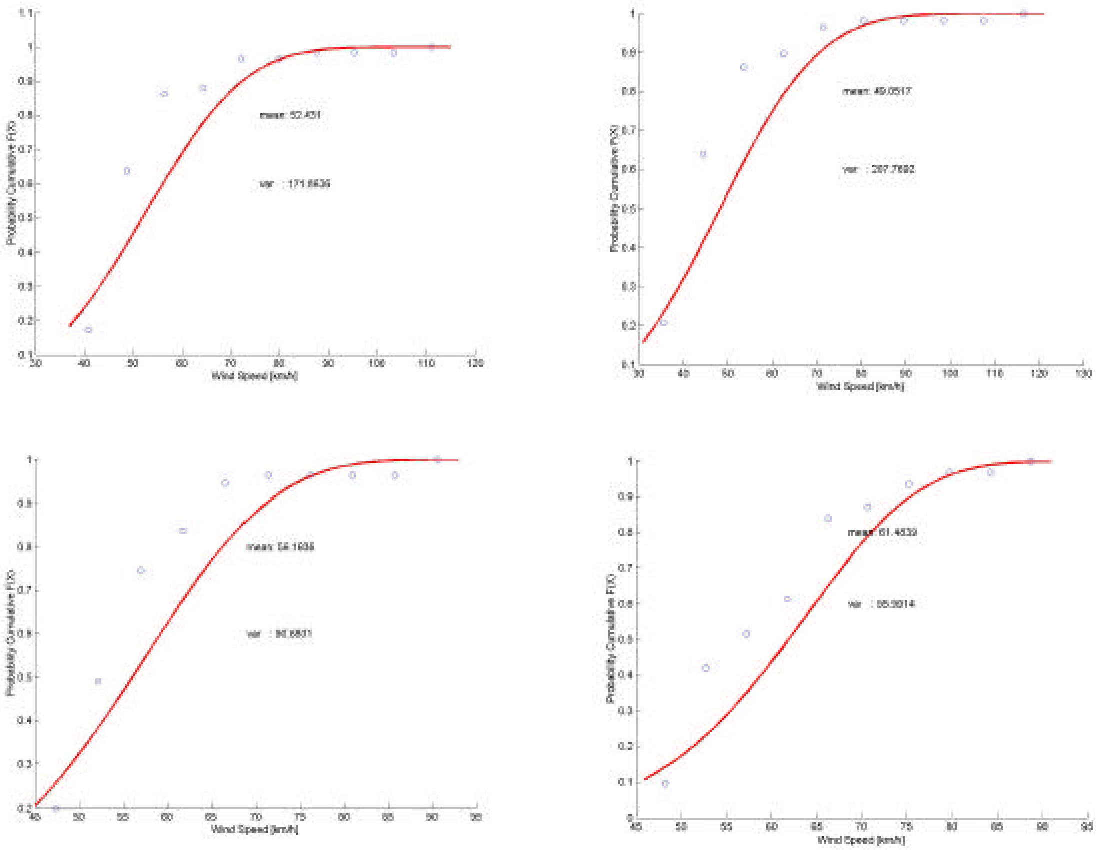


**Figure 3. The cumulative distribution functions of yearly rainfall, fitted with the Weibull distribution function.**

### 2.2 Extreme Wind Speed Distribution

In wind engineering, extreme value distribution of wind speed is normally estimated with the analysis of yearly-based data sets, which are also known as epochs (Simiu and Scanlan, 1996). Data set is described by $X \in X_1 \cup X_2 \cup \Lambda \cup X_n$, each of which has cumulative distribution function assumed to be $F_i$ ($i$=1, 2, …, $n$). When the cumulative distribution function of elements in the set are all same and independent as well, the distribution of the largest values of $n$ sets is then $F^n$ ($F_i$=$F$, $i$=1, 2, …, $n$), which is the extreme value distribution function.

From the data provided by BoM, the largest yearly wind speed recorded every 3 hours in Alice Spring, Darwin, Perth and Sydney can be obtained. The cumulative distribution function for the four locations are shown in Figure 4, and fitted with type II extreme value distribution function,

$$f_X(x|u,k) = \frac{k}{x}\left(\frac{u}{x}\right)^k e^{-(u/x)^k}$$
$$F_X^n(x|u,k) = e^{-(u/x)^k} \tag{1}$$

where parameters, $u$ and $k$, are given in Table 3. Also, from Eq.(1), the percent point function of type II extreme value distribution is given by,

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$$G_X(F_X^n) = u\left(-\log F_X^n\right)^{-\frac{1}{k}} \tag{2}$$

where

$$F_X^n = 1 - \frac{1}{N} \tag{3}$$

and $N$ is the mean return period or recurrence interval. For mean return period of 50 years, the 2 second gust wind speed are given in Table 4, where the multiplier 1.53 is used to convert 3 hourly speed to 2 second gust (ASCE, 1991).

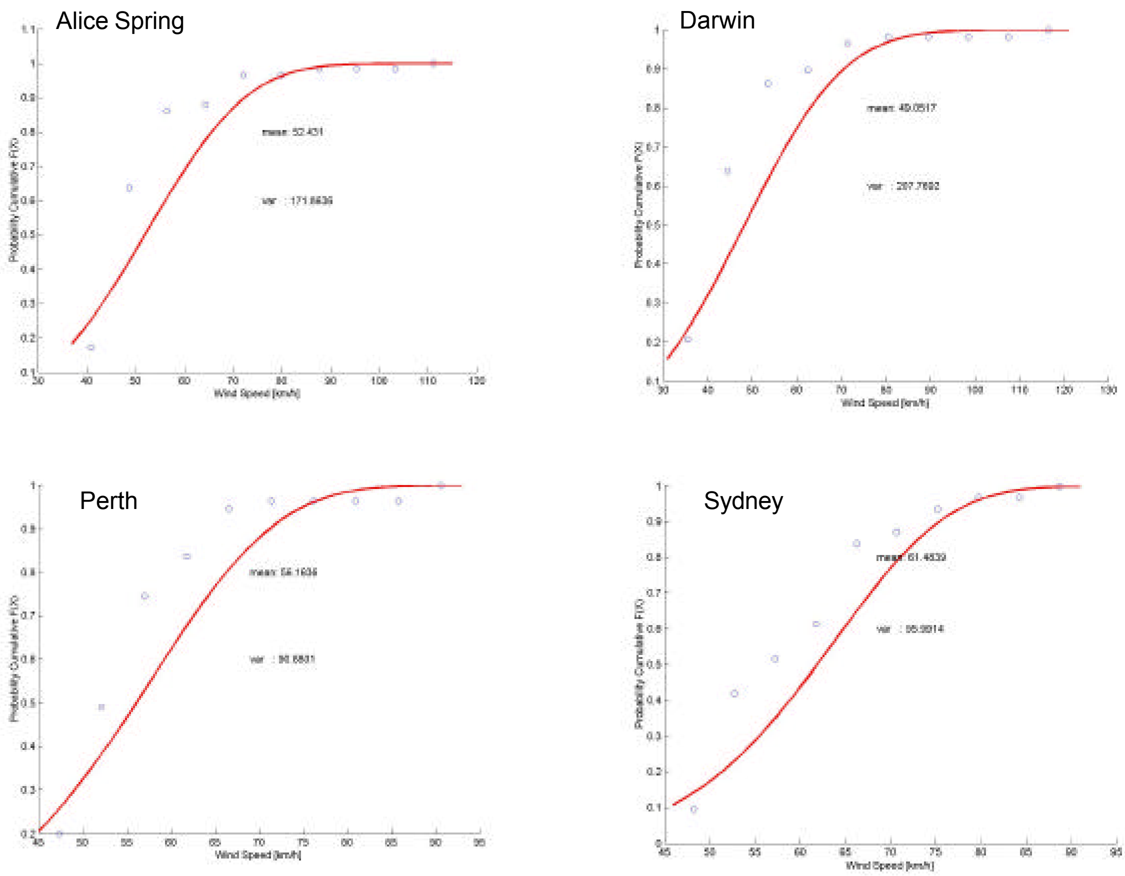


**Figure 4. The cumulative distribution function of extreme value of wind speed, fitted with type II extreme value distribution.**

**Table 3. Parameters of type II extreme value distribution functions fitted to the extreme wind speed from 3 hourly BoM records.**

| | Alice Spring | Darwin | Perth | Sydney |
|---|---|---|---|---|
| $u$ | 50.545 | 45.803 | 55.625 | 58.402 |
| $k$ | 8.946 | 7.5292 | 14.420 | 16.22 |

**Table 4. The extreme wind speed (km/h) at a mean recurrence interval of 50 years**

| Recurrence | Alice Spring | Darwin | Perth | Sydney |
|---|---|---|---|---|
| 50 years | 120 | 118 | 112 | 114 |

## 3 A MODEL FOR IN-GROUND DECAY OF TIMBER MATERIALS

As suggested by Leicester et al. (2000, 2001), the development of in-ground timber decay depth can be described by the following equation,

$$d_{decay} = \begin{cases} 0 & t \le t_{lag} \\ \dfrac{d_{20}}{20 - t_{lag}}(t - t_{lag}) & t > t_{lag} \end{cases} \tag{4}$$

where $t_{lag}$ is the lag time and $d_{20}$ is the decay depth at 20 years. For untreated heartwood, $d_{20}$ is given by,

$$d_{20,un,heart} = AI_{ig} \tag{5}$$

In Eq.(4) and (5), $t_{lag}$ and $A$ is described in Table 5.

**Table 5. Decay parameters for untreated heartwood timber.**

| Durability Class | A | $t_{lag,un,heart}$ |
|---|---|---|
| Class 1 | 3 | 6 |
| Class 2 | 7 | 4 |
| Class 3 | 11 | 2 |
| Class 4 | 30 | 1 |

$I_{ig}$ indicates the climate index, which is determined by,

$$I_{ig} = [f(R)]^{0.3}[g(T)]^{0.2} \tag{6}$$

where $f(R)$ and $g(T)$ are the functions of yearly rainfall and average temperature, and proposed to be,

$$f(R) = \begin{cases} 0 & \text{if } R \le 250\text{mm}, \\ 10[1 - e^{-0.001(R-250)}] & \text{if } R > 250\text{mm}, \end{cases} \tag{7}$$

$$g(T) = \begin{cases} 0 & \text{if } T \le 5^{o}\text{C}, \\ -1 + 0.2\text{T} & \text{if } 5^{o}\text{C} < T \le 20^{o}\text{C}, \\ -25 + 1.4\text{T} & \text{if } T > 20^{o}\text{C} \end{cases} \tag{8}$$

As a result of the assumption of Eq.(7) and (8), the occurrence probability of in-ground decay at the four locations is calculated, and results are then described in Table 6.

**Table 6. Occurrence probability of in-ground decay**

| Alice Spring | Darwin | Perth | Sydney |
|---|---|---|---|
| 0.544 | 1.000 | 0.977 | 0.997 |

## 4 .LIMIT STATE CRITERION

### 4.1 Loads Applied on Timber Poles

The utility pole can be considered as a cantilever beam with a circular cross section of diameter $d_p$. As shown in Figure 5, it is subject to a uniform pressure caused by wind loading as well as the concentration loads from tension of conductors that is induced by both wind load on the conductors and their own weight.

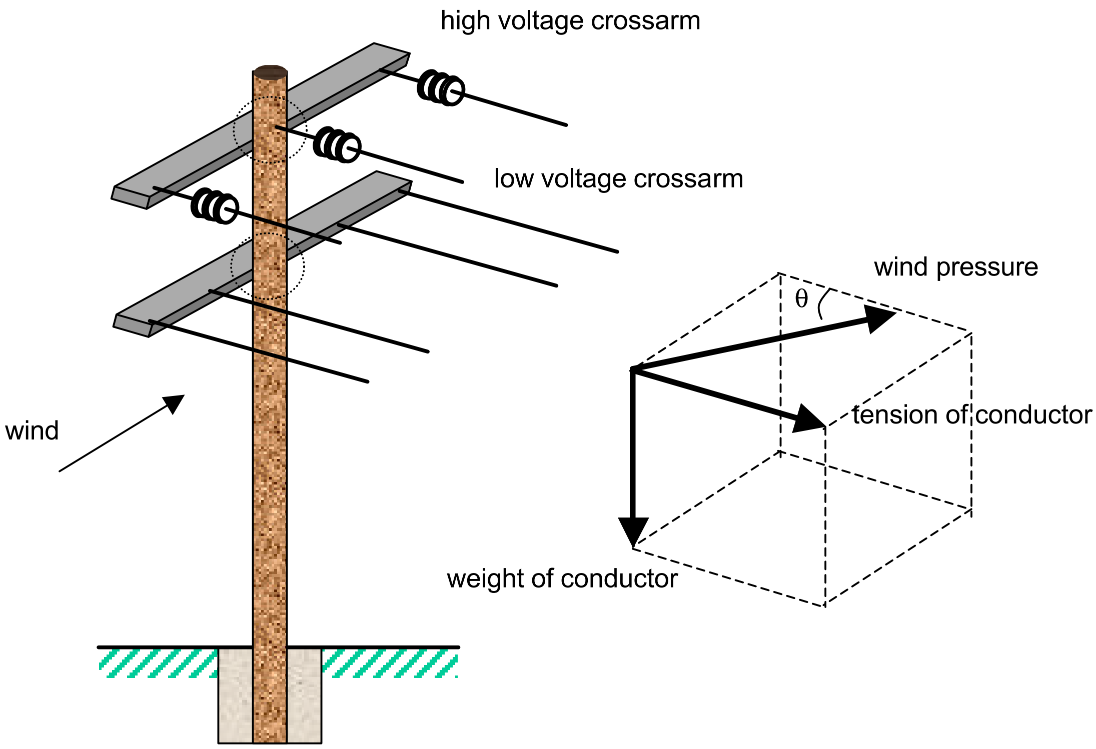


**Figure 5. A model of poles subject to wind loading and conductor weight.**

If we neglect the moment applied on the pole footing by the dead load due to the deflection of the pole, the moment are produced in terms of four parts,

- the wind pressure applied on the pole;
- the force, perpendicular to the direction of conductor cable, produced by wind loading on conductors;
- the force, parallel to the direction of conductor cable, contributed by the tension of conductors produced by wind loading; and
- the force, parallel to the direction of conductor cable, contributed by the tension of conductors produced by their own weight.

The wind pressure on the pole along the direction perpendicular and parallel to the conductor cables can be calculated, respectively, from,

$$\begin{aligned} F_{px} &= l_p d_p q \cos q \\ F_{py} &= l_p d_p q \sin q \end{aligned} \tag{9}$$

where $l_p$ is the length of the beam, and the wind pressure $q$ is determined by,

$$q = \frac{C_D}{2} r U^2 \tag{10}$$

and $C_D$ is the drag coefficient of the wind loading and equal to 1.20 for the beam with circular cross-section at a low Reynolds number (Simiu and Scanlan, 1996), $r$ is the air density, $U$ is the two-second gust wind speed discussed in the last section.

The concentrate force, produced by wind loading on conductors and applied on the poles along the direction vertical to the conductors, can be calculated by,

$$F_{cy} = \frac{1}{2} q d_c l_c \tag{11}$$

where $l_c$ is the terminating span of conductors, and $d_c$ is the diameter of the conductors. The concentrate force, produced by wind-load and weight induced tension, and applied on the pole along the direction parallel to conductors, can be calculated by,

$$F_{cx} = \frac{q_t d_c l_c^2}{8h_{mid}} \tag{12}$$

where $h_{mid}$ is the sag at the mid-span of conductor cables, and

$$q_t = q\sqrt{1+\left(\frac{r_c}{qd_c}\right)^2} \tag{13}$$

$r_c$ in Eq.(13) is the weight per length of conductor cables. As a consequence, the maximum of the moment applied on the foot of poles can be given by,

$$M_{\max} = \sqrt{\left(\frac{l_p F_{px}}{2} + \sum_i l^{(i)} F_{cx}^{(i)}\right)^2 + \left(\frac{l_p F_{py}}{2} + \sum_i l^{(i)} F_{cy}^{(i)}\right)^2} \tag{14}$$

where $i$ indicates the $i$-th crossarm, and $l^{(i)}$ and $F^{(i)}$ describe the related location and concentrated force.

### 4.2 Limit State Function

The limit state criterion of bending structural components can be described by,

$$g(MOR, d_{p-decay}, M_{\max}) = R(MOR, d_{p-decay}) - M_{\max} > 0 \tag{15}$$

Where $MOR$ is the Modulus of Rupture (MOR), $M_{max}$ is the maximum moment applied on structures, $R(MOR, d_{p-decay})$ is the bending failure resistance, dependent on initial $MOR$ and the decayed diameter. For timber poles with a circle cross-section, the resistance is expressed by,

$$R(MOR, d_{p-decay}) = MOR\frac{pd_{p-decay}^3}{64} \tag{16}$$

where $d_{p\text{-}decay}$ indicates the diameter of poles under decay attack, given by,

$$d_{p-decay} = \begin{cases} d_p & t \le t_{lag} \\ p\left[d_p - \dfrac{AI_{ig}}{20-t_{lag}}(t-t_{lag})\right] + (1\text{-}p)d_p & t > t_{lag} \end{cases} \tag{17}$$

and $p$ is the occurrence probability of decay at a specified location. From Eq.(16) and (17), obviously, the bending resistance is deteriorated with the occurrence of decay, and it declines with the service time.

## 5 LIFETIME SURVIVAL PROBABILITY OF TIMBER POLES

It is assumed that an extreme wind-loading event, $E$, has wind direction as well as occurrence within $(0, t_L]$ satisfying uniform probability distributions. As a result, the survival probability over period of $t_L$ is given by,

$$P[g(MOR, d_{p-decay}, M_{\max}; t_L) > 0] = \frac{1}{2pt_L}\int_0^{t_L} F_S^q\left(R(MOR, d_{p-decay}), M_{\max}; t\right)dt \tag{18}$$

where

$$F_S^q(t) = \frac{1}{2p}\int_0^{2p} F_S\left(R(MOR, d_{p-decay}), M_{\max}; t, q\right)dq \tag{19}$$

and $F_S$ is the survival probability at specified time and wind direction.

If there are $n$ independent wind-loading events, $E_i$ ($i$=1, 2, …, $n$), with an identical cumulative distribution function, $F_S$, in the period of $(0, t_L]$, the survival probability of timber poles is expressed by,

$$P[R(t_1) > 0 \,\mathrm{I}\, \ldots \mathrm{I}\; R(t_n) > 0] = \left[\frac{1}{2p\; t_L}\int_0^{t_L}\int_0^{2p} F_S\left(R(MOR, d_{p-decay}), M_{\max}; t, q\right)dq\; dt\right]^n \tag{20}$$

To consider a filtered Poisson process where the number of events, $n$, satisfying a Poisson process (Mori and Ellingwood, 1993), and assume $MOR$ with probability density function of $f_{MOR}(r)$, the lifetime reliability function then becomes,

$$L(t_L) = \int_0^{\infty} \exp\left( -\lambda t_L \left[ 1 - \frac{1}{2\pi\, t_L} \int_0^{t_L} \int_0^{2\pi} F_S\left( R(MOR, d_{p-decay}), M_{\max}; t, \theta \right) d\theta\, dt \right] \right) f_{MOR}(r) dr \qquad (21)$$

where λ is the occurrence rate of extreme wind loading. The failure probability can be calculated easily from,

$$F(t_L) = 1 - L(t_L) \qquad (22)$$

# 6 EXAMPLES TO PREDICT LIFETIME SURVIVAL PROBABILITY

## 6.1 Material Properties and Other Parameters

As an example, the electrical timber pole is shown in Figure 6. MOR and geometric of timber poles is indicated in Table 7. Parameters about conductors attached on the poles are described in Table 8. 2-second gust wind speed at the four locations follows Table 4, and the occurrence probability of decay in Table 6.

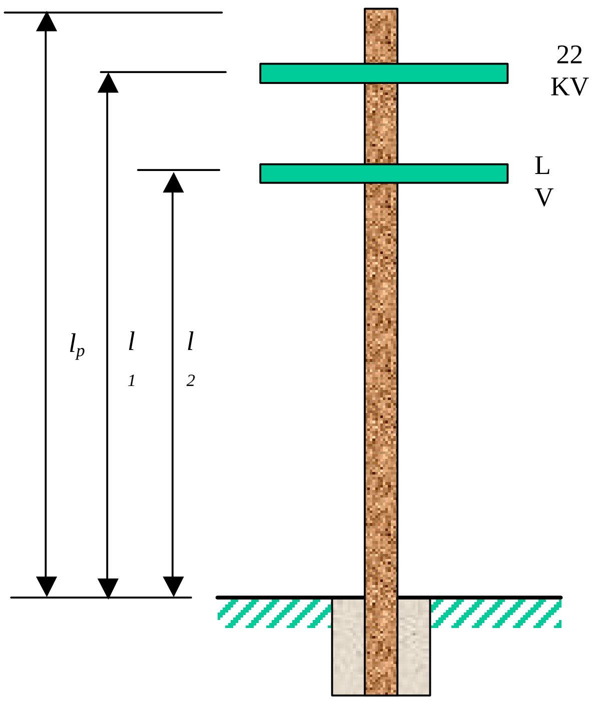


**Figure 6. Electrical timber poles with 22 KV and LV transmission lines**

**Table 7. Timber properties and parameters used in the simulation**

| **MOR [MPa]** | | Length $l_p$ [m] | **Diameter $d_p$ [m]** |
|---|---|---|---|
| **mean** | **Cov** | | |
| **160** | **0.1** | **10.2** | **0.35** |

**Table 8. Conductor properties and parameters used in the simulation**

| Voltage | Conductor | Number | Diameter $d_i$ [mm] | Linear Weight $r_i$ [N/m] | Location $l_i$ [m] | Span $l_c$ [m] |
|---|---|---|---|---|---|---|
| 1: 22 KV | 19/3.25AAC | 3 | 16.25 | 4.25 | 9.7 | 31 |
| 2: LV | 19/3.25AAC | 4 | 16.25 | 4.25 | 9.2 | 31 |

### 6.2 Bending Failure Resistance

Probability density function of the bending failure resistance can be calculated from Eq.(15) with the use of Monte-Carlo simulation. When MOR is assumed to follow a normal distribution function, the probability density function for 4 classes of timber materials in Darwin are shown in Figure 7.

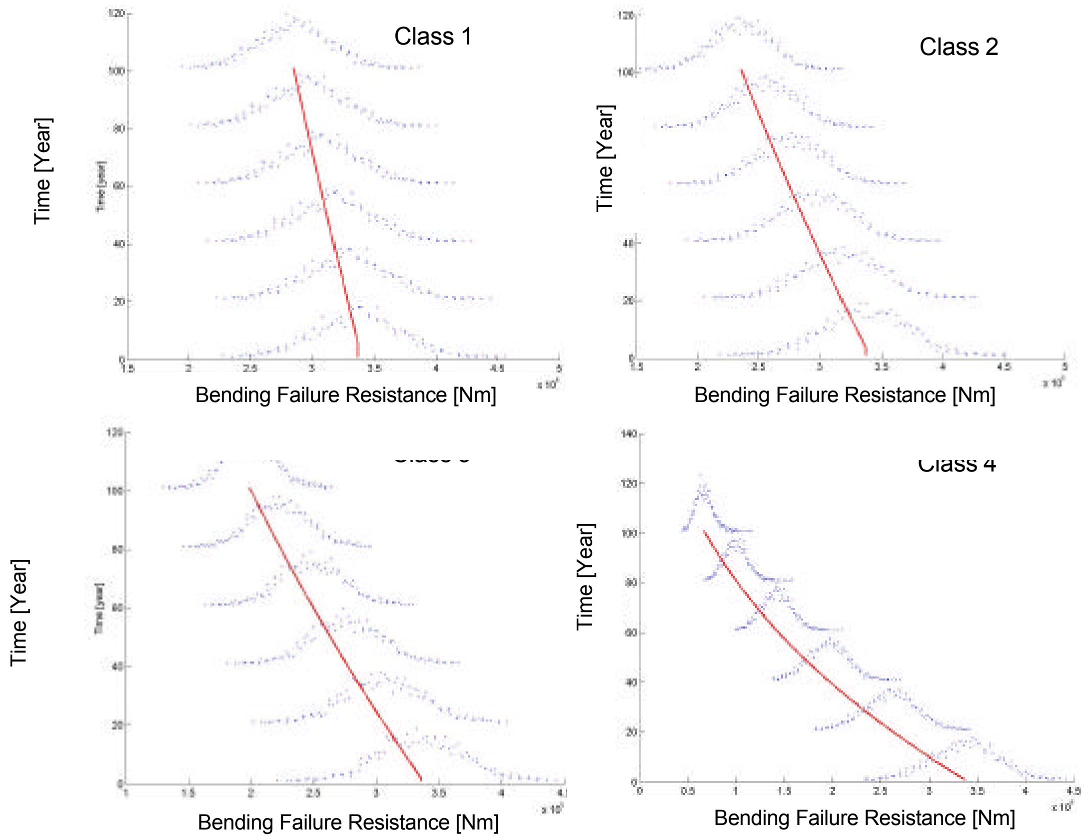


**Figure 7. The probability density function of bending failure resistance of 4 classes of timber materials. Location: Darwin.**

The probability density functions at other locations have a similar phenomenon. For Class 1 of timber materials, the deterioration is mostly shown in the shift of the distribution function to a lower level – reduction of the mean value. Whereas, for Class 4 of timber materials, the deterioration is displayed not only in the shift of the functions, but also in their shapes, which imply the change of the variation of the resistance.

### 6.3 Survival Probability

The conditional cumulative distribution functions of survival probability, $F_S^q(t)$, at specified time for 4 classes of timber poles at Alice Spring, Darwin, Perth and Sydney are shown in Figure 8. The lifetime survival probability, $L(t_L)$, is shown in Figure 9. As we can see from the figures, the poles made from class 4 timber materials are the most vulnerable to the decay attack. The timber poles at Darwin displays deterioration of reliability earlier than others for all 4 classes of timber materials. It is interesting to note that although the timber poles at Alice Spring shows earlier deterioration than Perth and Sydney due to the higher 2-second gust wind speed loading, the poles at the late two locations display more rapid deterioration than Alice Spring because of the relatively higher decay rate in both Perth and Sydney. Therefore, a higher design standard of timber poles at Perth and Sydney, particularly at Darwin, is required. It should be mentioned that lifetime survival probability plays a significant role in the optimisation of risk-based lifecycle maintenance strategies and management. This will be discussed in the later work.

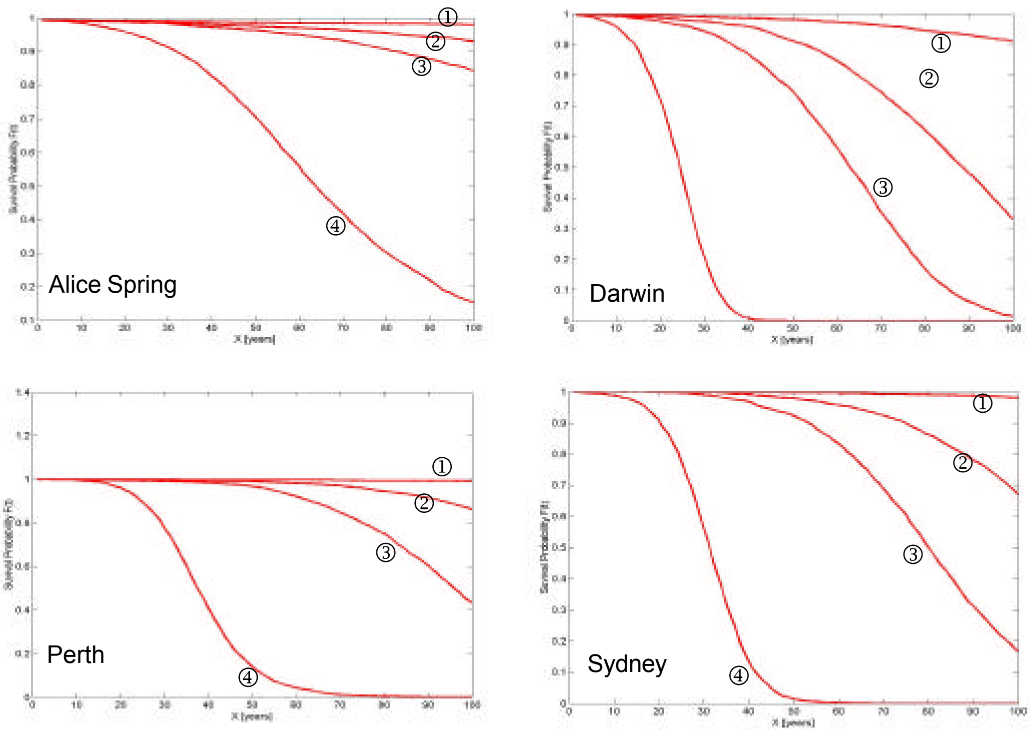


**Figure 8. Conditional cumulative distribution function of survival probability $F_S^q(t)$ with respect to the specified time for 4 classes of timber poles.**

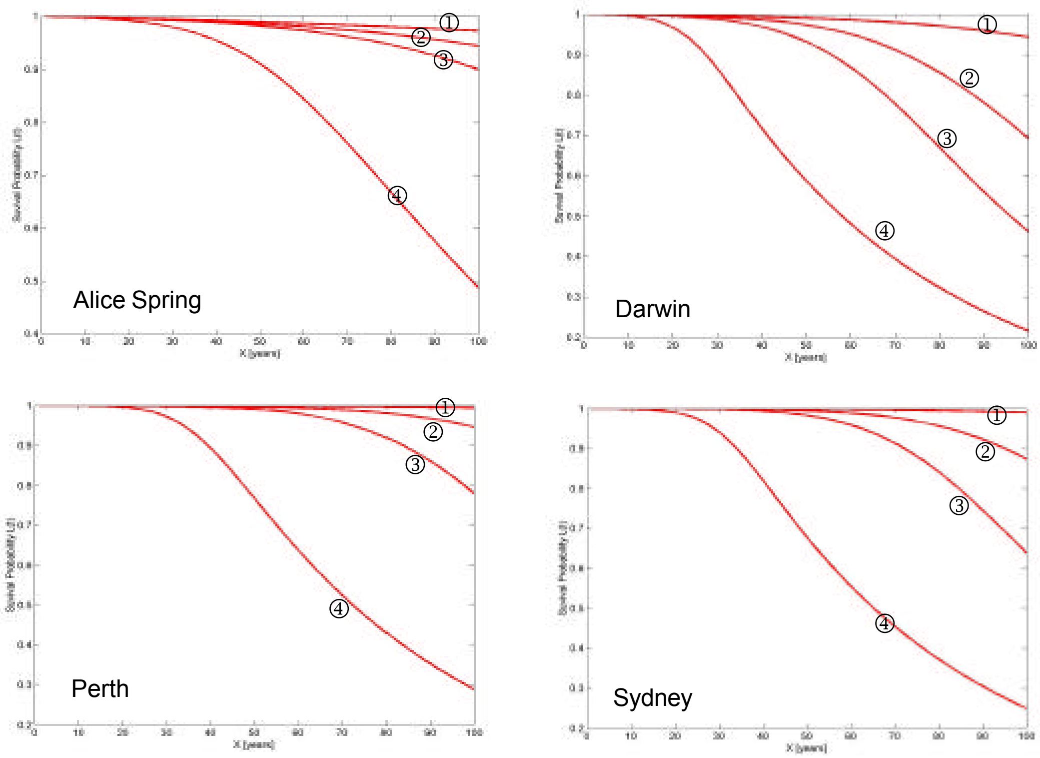


**Figure 9. Lifetime survival probability, L(tL), with respect to the time period for 4 classes of timber poles.**

## 7 CONCLUSIONS

As a consequence of growing aging and deteriorating timber poles, decisions have to be made in the management to determine if the deteriorated utility poles can maintain service above an acceptable reliability level without repair, with preventive maintenance at less cost to reduce deterioration growth, or with replacement to recover deteriorated reliability. The central point of the management is the reliability evaluation and prediction of existing or new utility poles at certain constraints. With the predicted lifetime reliability, optimal maintenance strategies can then be identified to minimise cost in a lifecycle view.

In this paper, a methodology to predict lifetime reliability of timber poles is proposed, considering the structural deterioration in the presence of in-ground decay. Climate data from BoM, including dry-bulb temperature, rainfall, wind speed are used to obtain local environmental conditions around timber poles and subsequently find their probability density function. With the results from BoM, an in-ground decay model as well as an extreme wind loading model are established. It is important to obtain information about poles network distribution and conductors attached onto the poles before calculating eventual loading applied on the poles. Based on the decay and loading models, a limit state function considering bending failure is derived. Finally, the lifetime survival probability is formulated, and applied to timber poles with 4 different classes of decay resistance.

It should be mentioned that the models and predictions were established based on the previous data record. When new information becomes available, all models and prediction can be updated by the use of Bayesian theorem. This will show in our late work.

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# Biodeterioration Of Gypsum - Counter Scrap Composites

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**Summary: This paper describes the results of a research study on the biodeterioration of a composite of gypsum and shoe counter scrap pieces. This composite was developed as part of a research program to study the feasibility of recycling of shoe counter scraps. Biodeterioration tests were made to identify some potential durability problems that would affect this material during its service life.**

**The main objective was to determine the influence of microbiological growth in the composite durability. A selection of fungi believed to have the ability to interact with these materials was made and the composite was exposed to them using two different methodologies: BDA cultures and tropical chamber cultures. Mechanical properties of the composite samples were measured before and after fungal growth.**

**This work complements a previous study on counter scrap pieces, where results indicated that some microbiological growth is expected, but no reduction on mechanical properties is to be found.**

**Keywords: Biodeterioration, composites, gypsum, shoe counter scrap.**

## 1 INTRODUCTION

This paper presents the results of a research on the durability of a gypsum and shoe counter scrap composite that was produced in a study of the feasibility of recycling shoe counter scrap. The counter is one of the components used in the manufacture of shoes, lending structural support to the shoe. It is produced by impregnating or laminating sheets with ethylene-vinyl-acetate based resins. The amount of waste material generated in the counter piece manufacturing process amounts to 13% to 25% by weight of raw material depending on the shape and model of counter piece produced. Estimates for the total amount of counter scrap generated in this region run at 80 metric tons monthly. According to manufacturers, currently technology available and costs do not make the industrial reuse of this material feasible.

The new construction material is obtained by adding shoe counter scrap, after cutting in a knife mill, to calcium sulfate hemihydrate matrixes. A new composite is then produced by adding scrap to the gypsum paste. The scrap behaves like a fiber and in this way improves some mechanical properties of the composite, with resulting changes in physical properties (Kazmierczak et al. 2000a).

However, in addition to determining the mechanical properties of the composite, it is necessary to study the phenomena affecting the durability of this material. This analysis requires the knowledge of the chemical properties and microstructure of this new material, as well as the knowledge of the characteristics of the environment this material will be exposed in order to allow the identification of any potential degradation agents. In this paper, the main objective was to determine the influence of microbiological growth in the composite durability, based on the mechanical properties of the composite samples before and after fungal growth.

## 2 MATERIALS

The gypsum matrix used was slow hardening calcium sulfate hemihydrate from the state of Pernambuco (Brazil). The hardening onset is after 22 minutes and it is over after 35 minutes.

Counter pieces are produced by impregnating sheets (IC) or laminating sheets (LC) with EVA-based resins. The two kinds of scrap materials are classified under the Brazilian Standard NBR 10004 as “class II waste – non-inert material”. The main difference between the two materials lies in the type of substrate.

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The tensile strength of the counter is anisotropic and ranges from 9.3MPa to 24MPa. Water absorption is lower than 3% on average for the laminated counter and ranges from 40 to 60% for the impregnated material. After the scrap has been ground, however, these values rise significantly and vary according to the particle size obtained. Specific gravity for laminated counter pieces ranges from 0.4 to 0.7g/cm$^3$.

Figures 1 and 2 show the two types of sheets.

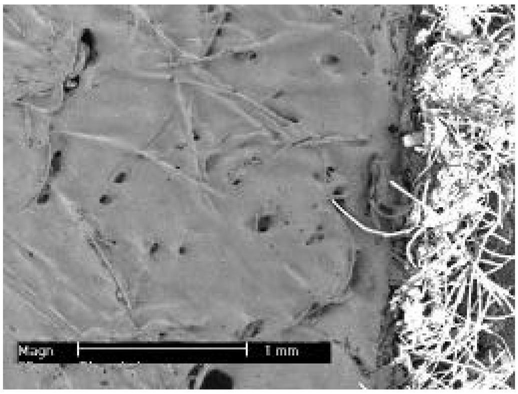


**Figure 1. SEM image of the impregnated counter scrap (IC).**

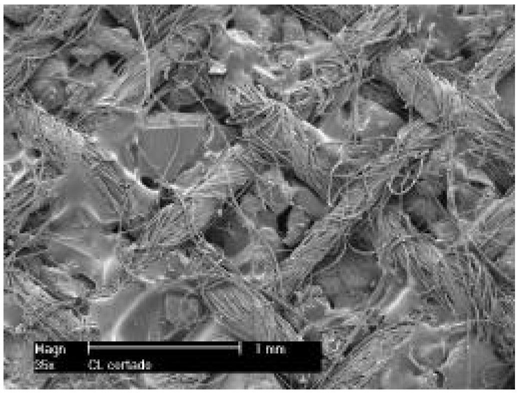


**Figure 2. SEM image of the laminated counter scrap (LC).**

The difference between these materials lies in the base material used in their manufacture. IC uses a non-woven fabric made up by randomly arranged fibers, while LC uses a woven film made up by intertwined threads that form a structuring base (fabric). After grinding, the counter displays a large number of non-oriented fibers due to the maceration of the impregnating resin and the original structure of the material is changed completely.

## 3 DURABILITY-RELATED ASPECTS

The materials used in the civil construction sector are usually exposed to complex degradation mechanisms, which can be external to the material (the environment around them) or determined by their structural characteristics. Any assessment of durability properties should be based on the precise characterization of the environment where this material will be used. In this study, the specification of requirements was made considering the use of this material in dry walls, which are not subject to harsh weather conditions.

As this material is made up by different components, the interactions between its components and the synergy of the degradation processes must be analyzed since these may result in new degradation mechanisms.

Previous studies (Kazmierczak et al. 2000b) determined the compatibility of the composite materials. This study was based on the chemical identification of the scrap constituents and their compatibility with gypsum. SEM analysis of one-year-old composite samples shows no visible signs of surface degradation of the fibers in the scrap surface, being a strong indication that the composite materials are compatible.

Figure 3 show an example of LC fibers involved by gypsum crystals with no signs of degradation.

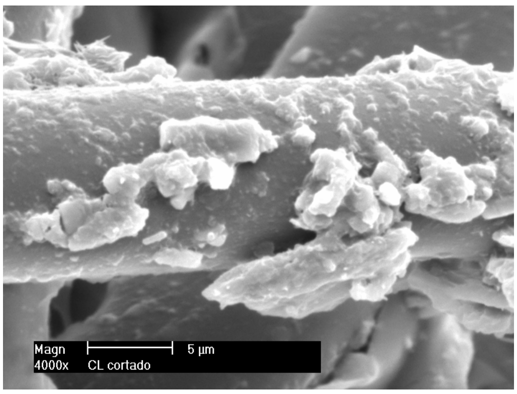


**Figure 3. SEM image of one-year-old LC fibers surrounded by gypsum crystals.**

Biodeterioration studies were carried out (Kazmierczak et al. 2000c) to identify the types of filamentous fungi that can grow in counters and their potential as deterioration agents, as well as to study alternatives to inhibit this process. The durability analysis was restricted to the counter scrap pieces because the development of a final product is not part of this study. Here, a selection of fungi believed to have the ability to interact with these materials was made, based on bibliographic sources and the identification of contamination agents in the counter scrap production process. The composite was exposed to them using two different methodologies: BDA cultures and tropical chamber cultures. The following fungi were used: *Aspergillus niger, Aspergillus flavus, Cladosporium, Gliocladium, Nigrospora, Penicillium, Rhizopus and Trichoderma.*

Stains were found in all the moist chamber samples, and the most representative species of fungi isolated were *Trichoderma* and *Penicillium*. Fungal growth can be explained by the increase in humidity and temperature levels combined with the increase in fungal dosage. It is believed that the staining was produced by the degradation of the resin or thermoplastic material and not of the fibers, which would be seen in the electron scanning microscope images. Samples of LC and IC are compared and a more expressive fungal growth is observed on the LC.

In the present study, the analysis is performed in the composite. The tests aim to check the effect of fungi on the tensile strength of fibers, which was not investigated before. The fungi used are the same listed above.

## 4 BIODETERIORATION STUDIES

### 4.1 Methods and materials

The study was performed using sets of four test specimens with scrap additions of 10 and 20% and water/gypsum ratio = 0.57. The test specimens were prepared with laminated counter scrap with a fineness modulus of 4.3. A series of control specimen tests, with no addition of counter scrap, was prepared.

The flexural strength in the composites was tested according to ASTM C78-94. Test specimens measuring 16cm x 4cm and 2cm thick were used to assess changes in the elastic properties of the composite as a result of fungal growth.

The fungi were isolated in a potato-dextrose-agar medium (PDA). The morphology of the vegetative and reproductive structures of the fungi was observed in an optical microscope and they were identified with the aid of a classification scheme (Barnett & Hunter 1972).

Samples of each composite were placed in a moist chamber with a 2cm deep soil layer and kept at $25\pm1^{o}$C at relative humidity of 70% with a photoperiod of 12h for 2 months. After this period, the flexural strength of the samples was determined.

### 4.2 Results and discussion

After the 8th day of incubation in the moist chamber, stains in the soil layer and in the samples produced by the fungi were observed. During the exposition, the surface area attacked by the fungi increased. At the end of the period of incubation all samples (even those without scrap) showed stains on the surface. The samples that showed more expressive fungal growth were those with scrap and w/g = 0,6.

The surface degradation in the control samples were used to estimate the extent of the incubation period in the moist chamber. It is believed that, considering the environment where this material will be used (in dry walls, not exposed to harsh weather conditions) this degree of degradation will not occur during the service life of these materials.

Concerning the flexural strength behavior of this material, preliminary studies (Kazmierczak et al. 2000a) demonstrate that the scrap added to a matrix has a fiber-like behavior and improves the ductility of the matrix and changes its flexural strength behavior. The addition-free gypsum matrix shows a sudden break, but the test specimens with the addition of scrap had considerable plastic deformation and became more ductile.

The flexural strength and mean results for each series of specimens tested are shown in Table 1.

Series with 0.6 and 0.8 water/gypsum ratios and no scrap addition (reference) displayed distinct performances. While fungal growth did not cause the flexural strength in the series with 0.8 water/gypsum ratios to change, the series with w/g = 0.6 showed an increase. This effect seems to be caused by an elongation of the gypsum hydration process during the period of incubation in the moist chamber.

**Table 1. Flexural strength results for each test specimen series before and after fungal growth**

| laminated counter samples composite | reference (fungi-free) | | after fungal growth | |
|---|---|---|---|---|
| | flexural strength (MPa) | average (MPa) | flexural strength (MPa) | average (MPa) |
| | 0.4 | | 1.0 | |
| control ( no scrap addition ) | 0.4 | 0.4 | 0.6 | 0.8 |
| w/g = 0.6 | 0.3 | | 0.7 | |
| | 0.4 | | - | |
| | 2.0 | | 2.4 | |
| control ( no scrap addition ) | 2.9 | 2.0 | 2.0 | 1.9 |
| w/g = 0.8 | 1.2 | | 1.4 | |
| | - | | - | |
| | 2.4 | | 1.3 | |
| scrap addition: 10% | 1.9 | 2.0 | 2.4 | 2.0 |
| w/g = 0.6 | 2.2 | | 1.9 | |
| | 1.6 | | 2.2 | |
| | 1.5 | | 1.1 | |
| scrap addition: 10% | 1.6 | 1.5 | 1.3 | 1.3 |
| w/g = 0.8 | 1.5 | | 1.3 | |
| | 1.4 | | 1.3 | |
| | 2.3 | | 2.3 | |
| scrap addition: 20% | 2.3 | 2.2 | 2.5 | 2.2 |
| w/g = 0.6 | 2.0 | | 1.9 | |
| | 2.2 | | 2.1 | |
| | 1.3 | | 1.3 | |
| scrap addition: 20% | 1.3 | 1.4 | 1.3 | 1.3 |
| w/g = 0.8 | 1.4 | | 1.3 | |
| | 1.5 | | 1.2 | |

The effect of fungal growth on the flexural strength of the composite can be seen in Figure 4. Flexural strength decreases in the series with w/g = 0.8 but shows no change in the series with w/g = 0.6. The different behavior in the series with different w/g relationship can be explained by the fact that the control samples with w/g = 0.6 showed increased flexural strength.

The difference in flexural strength after fungal growth was not significant (the highest decrease in strength was 13%).

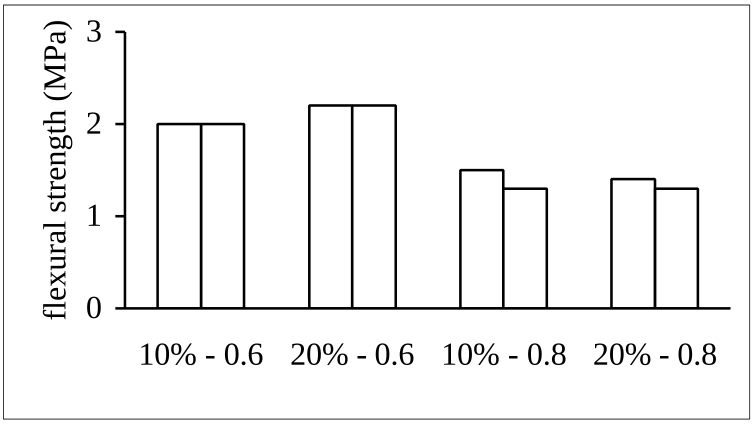


**Figure 4. Mean values of flexural strength of the series tested before (left) and after (right) incubation in the moist chamber.**

## 5 FINAL REMARKS

The assessment of the durability of the new composite made with gypsum and counter scrap was based on the knowledge of the chemical properties and microstructure of this new material, as well as the knowledge of the characteristics of the environment to which this material will be exposed.

The main objective of this study was to determine the influence of microbiological growth in the durability of this composite, based on the flexural strength of the composite samples before and after fungal growth.

Results indicate that some microbiological growth is expected, but no significant reduction on mechanical properties is to be found.

## 6 ACKNOWLEDGEMENTS

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# Durability And Performance Characteristics Of Recycled Aggregate Concrete

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**Summary: In recent years, the recycling of concrete to produce aggregates suitable for non-structural concrete applications is emerging as a commercially viable and technically feasible operation. In this paper, we report on performance and durability tests carried out to determine the fresh and hardened properties of concrete made with commercially produced coarse recycled concrete aggregate and natural fine sand. Test results indicate that the difference between the characteristics of fresh and hardened recycled aggregate concrete and natural aggregate concrete is relatively narrower than reported for laboratory-crushed recycled aggregate concrete mixes. For concrete without blast furnace slag, having similar volumetric mix proportions and workability, there was no difference at the 5% significance level in concrete compressive and tensile strengths of recycled concrete and control normal concrete made from natural basalt aggregate and fine sand. Water absorption rates and carbonation of recycled concrete and reference concrete were comparable. However, the abrasion loss of recycled aggregate concrete made with ordinary Portland cement increased by about 12% compared to normal concrete, while the corresponding drying shrinkage was about 25% higher at one year.**

**Long-term performance results generated from field case studies involving the construction of a 120 m long bicycle/footpath with a 40 m recycled aggregate concrete (RC) segment are discussed. The paper further discusses observed fresh and hardened concrete properties of both recycled and conventional aggregates, and examines specific implications for satisfactory performance of the recycled product for recreational and non-structural construction applications.**

**Keywords: Aggregates, compressive strength, concrete, durability, recycling.**

## 1 INTRODUCTION

Intensive global R&D effort conducted over the last few decades demonstrates the potential viability of recycled concrete aggregate (RCA) as a concreting material (Hansen 1992; Sri Ravidrarajah *et al*. 1987; Sagoe–Crentsil *et al*. 1996). Notwithstanding what has been achieved, there still exists a scarcity of durability performance and engineering data on the serviceability of construction works incorporating recycled aggregates. This situation has arisen because, until recently, the use of crushed concrete derived from building demolition has been restricted to granular sub-base layers in road pavements, and drainage or excavation fill applications, rather than the production of new concrete. However, the emergence of new crusher technologies, including rubble processing, grading and aggregate washing equipment, together with tighter regulation of the recycling industry (Topcu 1997), have contributed to significant improvements in the quality of recycled concrete aggregate products.

While studies on the engineering properties of concrete made with laboratory-crushed recycled concrete aggregate abound (Frondistou–Yannas 1980; Hansen & Narud 1983), only limited data is available on commercial grade recycled concrete aggregate, including concrete mix proportions, fresh concrete performance and durability characteristics. Technical consideration for the use of recycled aggregates in premix concrete production, however, is strongly dependent on achieving satisfactory fresh and hardened concrete performance. Thus, this paper evaluates aspects of recycled concrete durability and field performance based on commercial production of N25 (25 MPa) grade premix concrete for recreational construction works.

## 2 LABORATORY TESTS

A single source of commercially graded unwashed coarse RCA and natural fines was used in all recycled concrete mixes. The natural coarse aggregate used in the reference mixes was a nominal 14 mm crushed basalt. The grading of the basalt and the coarse RCA conformed to the requirements of AS 2758.1 (Standards Australia 1998), as shown in Table 1. A fine to coarse aggregate ratio of 46:54 was kept constant throughout the test program for all concrete mixes. The RCA was batched in the as-received state. Details of the physical properties of both aggregates are shown in Table 2.

Ordinary Portland cement (Type GP) and blast furnace slag cement (Type GB) comprising 65/35 Portland cement/blast furnace slag were used in the investigations. The slag blend was used partly to assess the possible improvements in fresh concrete cohesion, workability and concrete performance.

## 2.1 Concrete mixes

Several preliminary trial mixes were prepared to evaluate water requirements for a nominal 25 MPa concrete. Mix proportion data is given in Table 3. The unit water content of the concrete was corrected for free moisture in the aggregates. However, the recycled aggregate was presaturated for 10 minutes in the mixer and brought to room temperature prior to mixing the concrete. The mixes were proportioned to have a nominal binder content of 240 kg/m$^3$ for mixes C0912A, C0912B and C1212A, while mix C1212B contained an additional 5% cement to assess the effect of increased cement content. Slag cement was used in mix C1212A.

The water/binder ratio (W/B) of all mixes was adjusted to achieve comparable consistency and a nominal slump of 80 ± 15 mm. A reduction in the water requirement was attained by using a lignosulfate-based water-reducing admixture at nominal doses recommended by the manufacturer. The recycled concrete mixes contained 100% recycled aggregate and natural fine sand, while the normal concrete mixes contained all natural coarse and fine aggregates.

**Table 1. Sieve analysis of coarse RCA and basalt aggregate**

| *Coarse aggregates* | *Percentage of mass passing through sieve (mm)* | | | | | | |
|---|---|---|---|---|---|---|---|
| *Maximum size (mm)* | *19* | *13.5* | *9.5* | *6.7* | *4.75* | *2.36* | *150 mm* |
| RCA | 100 | 91.4 | 28.7 | 7.6 | 5.4 | 0.2 | 0.5 |
| Basalt | 100 | 84.0 | 43.7 | 5.6 | 2.1 | 1.0 | 0.2 |

**Table 2. Properties of RCA and basalt aggregate**

| *Property* | *RCA* | *Basalt* |
|---|---|---|
| Aggregate crushing value (%) (AS 1141.21) | 23.1 | 15.7 |
| Bulk density (kg/m$^3$) (AS 1141.6) | 2394 | 2890 |
| Water absorption (%) (AS 1141. 6) | 5.6 | 1.0 |
| Impurity level (%) (AS 1141. 32) | 0.6 | <0.1 |
| LOI (%) | 4.9 | 1.3 |

**Table 3. Mix designation and mix details of concrete specimens**

| *Mix designation* | *Binder loading (kg/m$^3$)* | *Binder type* | *W/B* | *Slump (mm)* | *Wet density (kg/m$^3$)* | *Entrapped air content (%)* | *Coarse aggregate* |
|---|---|---|---|---|---|---|---|
| C0912A (*OPC/basalt*) | 242 | Type GP | 0.76 | 90 | 2466 | 2.4 | Basalt |
| C0912B (*OPC/recycled*) | 240 | Type GP | 0.73 | 75 | 2335 | 2.4 | Recycled |
| C1212A (*Slag/recycled*) | 238 | Type GB | 0.74 | 95 | 2321 | 1.8 | Recycled |
| C1212B (*OPC+5%/recycled*) | 254 | Type GP | 0.70 | 80 | 2335 | 2.3 | Recycled |

Specimens were cast from each mix to assess compressive strength, drying shrinkage, expansion, splitting tensile strength and abrasion resistance. Unless otherwise specified, all specimens, upon their removal from the moulds, were stored under standard moist curing conditions of 23°C and >95% RH until required for testing. Hardened concrete testing was performed in accordance with the requirements of AS 1012.

### 2.2 Laboratory tests – results and discussions

Based on visual inspection, the surface texture of the RCA appeared characteristically grainy compared to the basalt. This is partly a result of the residual cement mortar attached to the RCA particles, and is also responsible for the characteristic lower density and corresponding higher aggregate crushing values compared to the basalt aggregate (Hansen 1992; Frondistou–Yannas 1980).

As indicated in Table 2, RCA also has a comparatively high water absorption value at 5.6%. Although the water absorption of the recycled aggregate is relatively high compared to the reference basalt aggregate, there was no difficulty in achieving the desired consistency and subsequent compaction of concrete.

Compressive strengths were determined on concrete cylinders continuously stored under moist conditions for up to 365 days. As shown by the mean compressive strength results plotted in Fig. 1, there is no significant difference between the strength of Portland cement concretes, as a function of aggregate type, for the grade of concrete investigated.

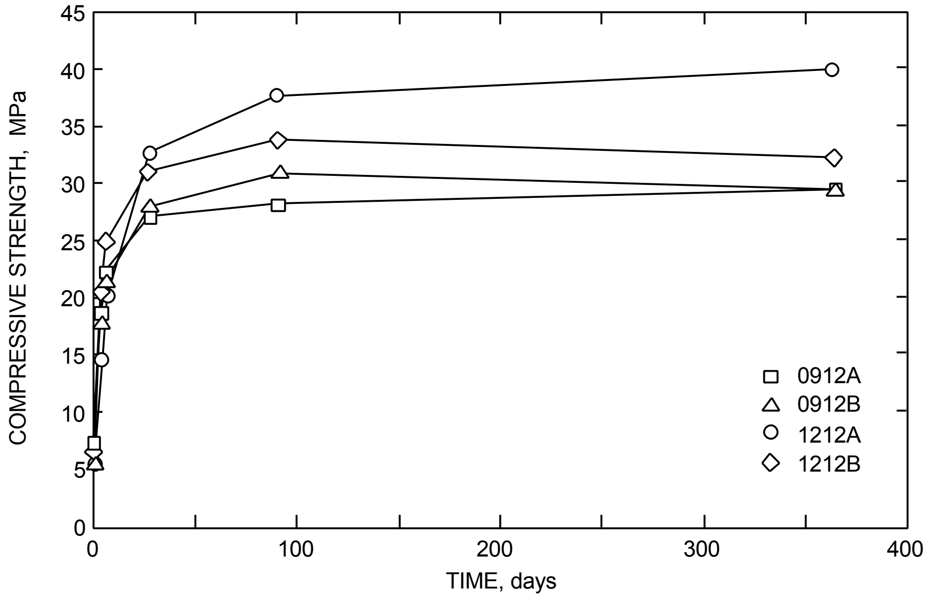


**Figure 1. Development of concrete compressive strength with age**

The corresponding increase in strength gain of the slag cement concrete from 20.2 to 32.6 MPa between 7 and 28 days is significantly higher than the equivalent nominal 6 MPa average increase for the Portland cement concretes. As noted in Fig. 1, the strength gain for recycled concrete made with Portland cement remains virtually unchanged at the 5% significance level beyond 28 days.

As shown in Table 4, the splitting-tensile/compressive strength ratios of recycled concretes are comparable to results obtained for conventional concretes made with natural aggregates. This ratio provides an indication of the resistance of a concrete to tensile strain and is dependent on type and size of coarse aggregate particles, voids in the concrete, and curing and test conditions. Calculated splitting-tensile to compressive strength ratio values ranged from 0.89 to 1.21 for recycled concrete, compared to 0.8 to 1.4 for standard aggregate concretes.

The variation of drying shrinkage strain with time for both recycled and reference concretes is shown in Fig. 2. The drying shrinkage of test specimens increased with time and stabilised at about 91 days, following similar trends reported by several researchers (Sri Ravindrarajah *et al*. 1987; Topcu 1997) for laboratory-crushed recycled concrete, although the absolute shrinkage values are slightly lower in the present case. While both natural and recycled aggregates display similar trends with regard to the rate of shrinkage, strains associated with recycled concrete made with slag cement at 365 days are over 35% higher than the reference mix, and closer to typical published values of 30–70% (Hansen 1992; Sri Ravindrarajan *et al*. 1987). In contrast, only a 15% increase in shrinkage strain is obtained for Portland cement specimens for the same curing conditions. However, there appears to be a much less significant effect on drying shrinkage as a result of the 5% difference in cement content.

**Table 4. Ratios of splitting-tensile ($f_t$) and compressive strength ($f_c$) of concretes**

| *Concrete mix* | $f_t/f_c$ | | |
|---|---|---|---|
| | *7 days* | *28 days* | *365 days* |
| C0912A (*OPC/basalt*) | 1.19 | 1.07 | 0.99 |
| C0912B (*OPC/recycled*) | 1.19 | 1.20 | 0.94 |
| C1212A (*slag/recycled*) | 1.21 | 0.98 | 0.96 |
| C1212B (*OPC+5%/recycled*) | 1.08 | 0.97 | 0.89 |

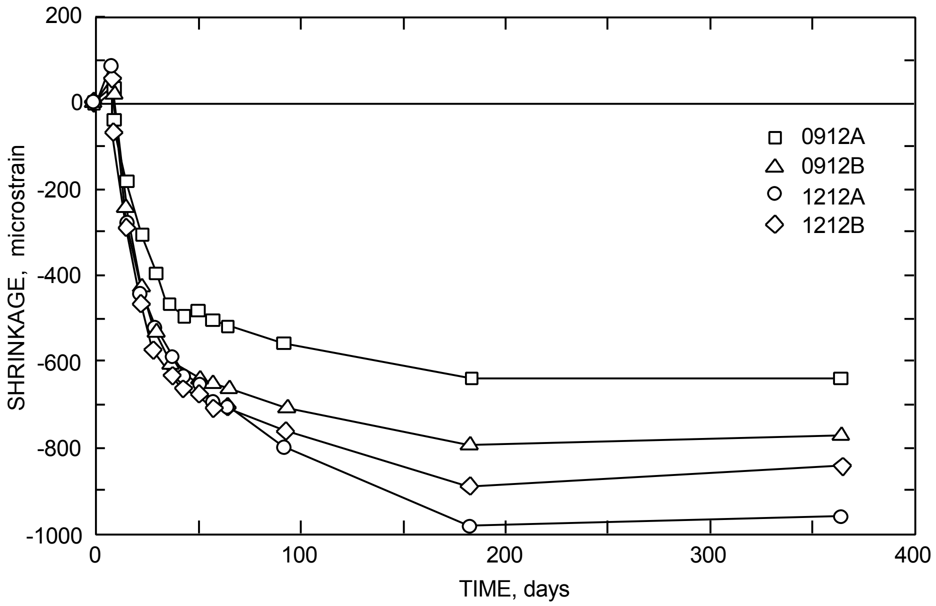


**Figure 2. Relationship between drying shrinkage strain of recycled and reference concrete with time**

It is evident from Fig. 2 that RCAs display higher drying shrinkage values compared to the reference normal concrete mixes, possibly partly due to the lower restraining capacity of recycled aggregate particles compared to basalt. Currently, AS 3600 (Standards Australia 1994) recommends a basic shrinkage limit of 700 microstrain at 56 days.

## 3 FIELD TRIAL

### 3.1 Project description

The field trial involved the construction of a bicycle/footpath facility located at Duncans Road, Werribee South, Victoria, Australia. The project involved the construction of a 120 m long, 2 m wide and 100 mm thick bicycle/footpath, a section of which is shown in Fig. 3. A segment, about 40 m long, was constructed with recycled aggregate concrete(RC), and normal grade concrete(NC) for the remainder. Normal base course preparation was employed. Both concrete types contained polypropylene fibres. No steel mesh was used. The Council supplied both RC and NC from a commercial batching plant and the contractor completed placement of both types of concrete over a two-day period. Both concrete types were formulated to the same mix design and were supplied by a local premix concrete producer.

Concrete compressive strength measurements and shrinkage tests were carried out on in-situ cored samples, as well as field- and laboratory-exposed specimens made from the supplied concrete batches. The specified concrete strength was 25 MPa.

### 3.2 Observations

The measured concrete slump was 75 mm and 115 mm for the RC and references (NC) mixes respectively. The air temperature on the day of casting for the RC was 11°C and 18°C for NC. Placement and finishing for both types of concrete presented no difficulties for the contractors.

### 3.3 Field trials – results and discussion

#### 3.3.1 Compressive strength

As shown in Figs 4 and 5, the 28-day fog cylinder compressive strengths of RC and NC concrete were 27.0 MPa and 28.1 MPa respectively – well above the specified 25 MPa cylinder strength. The corresponding 28-day strengths of field-cured concretes with a length to diameter ratio of ~1:1 were 20.1 MPa for RC and 23.2 MPa for the NC concrete. The core sections were taken from experimental concrete test strips cast adjacent to the path, as shown in Fig. 3. Concrete strengths at six months were comparable.

Figure 3. Completed footpath showing section of experimental test strip

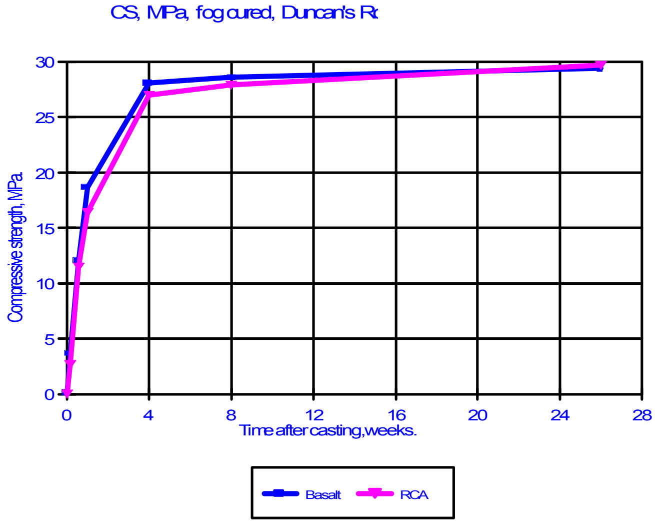


Figure 4. Fog-cured compressive strengths of RC and NC concretes

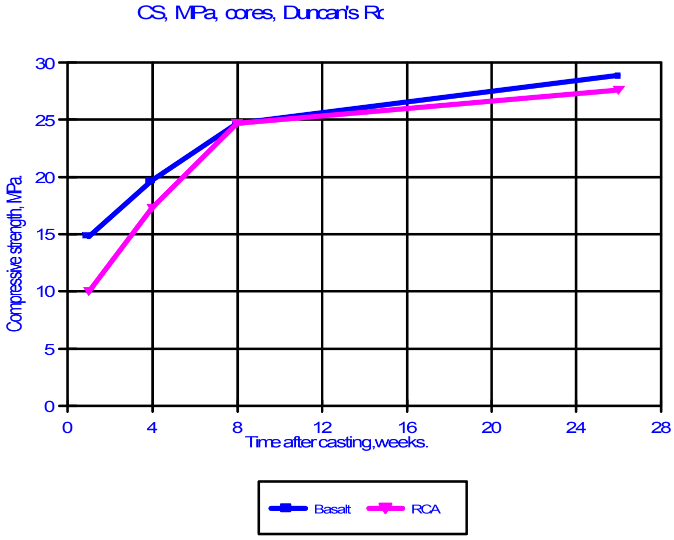


Figure 5. Core compressive strengths of RC and NC concretes

The 26-week compressive strength development of 95 mm diameter cored sections was of the same order for NC and RC concretes. It is conceivable that the difference in ambient temperatures at the time of casting may be partly responsible for the marginally lower initial strength of RC specimens, particularly given that Type GB cement was used.

3.3.2 Dimensional stability

Shrinkage data is shown in Figs 6 and 7. A separate set of specimens was stored in a conditioned room at 23°C and 50% RH, after 7 days of fog cure. Measured shrinkage of field-exposed specimens 56 days after casting are–0.122% for RC and –0.098% for NC respectively. Equivalent results of field-exposed samples were –0.021% and –0.020% respectively for RC and NC specimens.

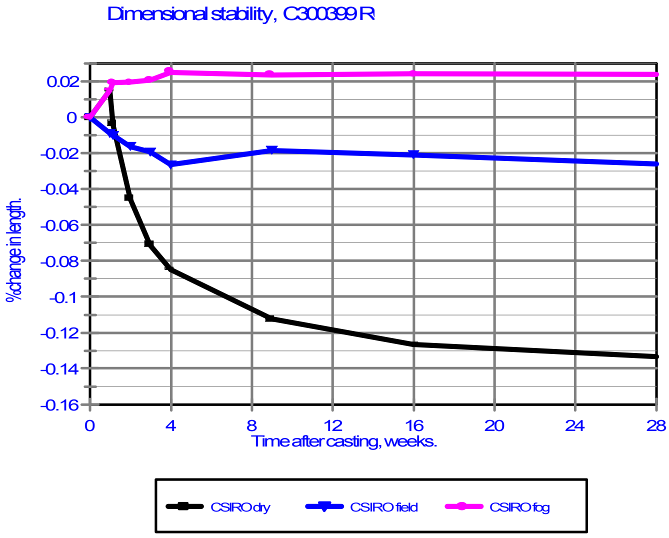


**Figure 6. Drying shrinkage of RC concretes**

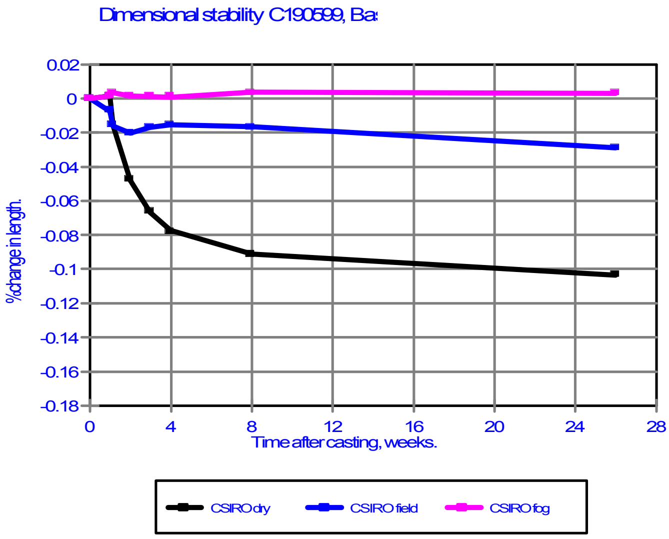


**Figure 7. Drying shrinkage of NC concretes**

# 4 CONCLUSIONS

The satisfactory demonstration of a real-life constructed facility, as reported in this paper, provides a preliminary indication of the technical performance of recycled aggregates, whilst promoting consumer awareness of waste minimisation and recycling in the construction sector. The results of the trials specifically indicate the potential for partial or full replacement of virgin aggregates with recycled material in non-structural construction applications. The preliminary findings of the project further indicate the following.

1. The quality of commercially produced coarse RCA and the performance of premix recycled concrete are, possibly, at a level where uniform concrete quality is achievable.
2. The same equipment and procedures used for concrete containing conventional aggregate may be used to batch, mix, transport, place and finish recycled premix concrete, including curing requirements. Initial trialing of recycled concrete mix designs are, however, recommended should there be a need for appropriate minor adjustments to achieve the desired performance.
3. Over the six-month test period, recycled premix concrete typically exhibits comparable characteristics similar to those of conventional concrete, and generally satisfies nominal specification requirements for concrete performance.

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4. Similar performance trends exists in RC and NC concretes for both the fresh and hardened states, including the response to conventional mix design methods, except for differences in measured drying shrinkage values for two of the three grades of concrete investigated, i.e. 15 and 25 MPa concretes.

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# Development Of Guide Specifications For Recycled Aggregates In Concrete Construction

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**Summary: Recycling of construction materials is growing along with the demand for recycled materials. However, this growth is often constrained by specifiers' insufficient knowledge of material performance, low awareness of benefits, and perceived risks. Until recently, in Australia, the use of crushed concrete derived from building demolition has been restricted to granular sub-base layers in road pavement construction and drainage, or excavation fill applications. However, improved crusher technologies, rubble screening and aggregate washing, and tighter regulation of the recycling industry, have contributed to significant improvements in the quality of recycled concrete products. Consumer acceptability of recycled materials, however, largely depends on products being technically suitable, cost competitive and meeting environmental impact requirements.**

**This paper discusses the proposed Guide specification document which will cover existing technical information and potential uses of recycled concrete and masonry waste into a structured format, required for specification guidelines. This includes information on recycled material properties and performance, and examines technical and market considerations of construction and demolition (C&D) waste recycling, based on field trials of premix recycled concrete. Different sources of aggregate batches are assessed in relation to existing Australian Standards for natural aggregates, in particular AS 1141, to establish conformance of recycled products. The proposed Guide specification document will provide engineers with product specification information and the tools required for conventional design with graded recycled C&D waste material.**

**Keywords: Concrete, guidelines, recycled aggregate, specifications.**

## 1 INTRODUCTION

The development a national Guide specification document on recycled concrete and masonry aims at providing engineers with product specification information and the tools required for conventional design with graded recycled construction and demolition (C&D) waste material. The document broadly outlines general specification requirements for recycled concrete and masonry materials, based on laboratory investigations and detailed local field performance evaluations of civil and municipal construction works involving recycled concrete aggregates.

Current national experience and knowledge regarding the use of these materials not only vary from material to material, but also in the way in which they are used. Furthermore, the availability of specification guidelines specifically for secondary and recycled materials is limited. However, in recent years, the gradual shift towards performance-based specifications has given rise to amendments of existing codes that also permit the use of all materials irrespective of their source, provided they comply to required specified performance rather than the conventional prescriptive requirements. Given that the recycling of construction materials is growing along with the demand for recycled material, it is important for specifiers to gain knowledge of material performance and an awareness of the benefits and perceived of risk of recycled construction materials.

## 2 PROJECT OBJECTIVES

The document provides general guidance to those who have an interest in using or increasing their understanding of recycled concrete aggregate (RCA) and masonry materials in construction applications.

These guidelines cover recycled material performance, limitations, and product testing and evaluation compliance requirements when considering the use of these materials. It is intended to assist suppliers, specifiers, regulators and end-users of recycled aggregate materials to gain some understanding of key material properties, by providing relevant information for determining the suitability of recovered material for use in selected civil, recreational and municipal construction applications. It is envisaged that manufacturers of recycled concrete and masonry products can produce material of suitable specifications for its intended use, based on standard crushing and grading processes for target applications.

The extensive survey of local applications of recycled concrete highlights case studies where recycled aggregate products either meet or fail to satisfy service performance requirements.

## 3 CONCRETE RECYCLING

Recycled concrete is basically old concrete that has been demolished and removed from foundations, pavements, bridges or buildings, and crushed into various size fractions for reuse. The feedstock material often includes small quantities of natural rock and construction masonry. Generally, recycled products can be produced from all but the poorest quality clean feedstock through a series of processing stages involving crushers, screens and devices for removing foreign material.

Material recycling in construction has been practised for many years now. However, there are several emerging issues relating to material specifications, testing and compliance protocols, characterisation procedures, design practice and material durability that need to be established. Whereas industry standard quality assurance procedures and product performance protocols are rapidly evolving, the need exists for tighter regulation given the diversity of feedstock sources and variations in the mode of aggregate production.

A high proportion of conventional demolition waste, particularly the fraction derived from concrete, brick and tile, is well suited to being crushed and recycled as a substitute for newly quarried (primary) aggregates. These materials are currently widely used in lower grade applications, most notably engineering fill and road sub-base applications. The use of such recycled concrete aggregates in new concrete is much less common, and technically much more demanding.

For these selected construction applications, recycled materials have the potential to partially displace equivalent volumes of primary aggregates. Preservation of non-renewable virgin aggregate resources, in turn, reduces the pressure on increasingly scarce landfill space.

In recent years, well-processed recycled products produced from good quality feedstock material have been demonstrated to comply with most test requirements for conventional materials for a range of applications. Several of these quality recycled aggregate products, which meet the same or equivalent performance specifications, are in current use in place of primary aggregates. The use of recycled products is therefore expected to increase with time and meet lesser discrimination in the market place.

Test procedures for virgin materials are currently used to characterise generic fundamental properties of recycled aggregates, as listed below. Data generated from such tests are useful in assessing overall performance of recycled aggregates compared to conventional materials, particularly for road base and new concrete applications:

- grading, particle shape and surface texture;
- specific gravity;
- moisture absorption;
- aggregate crushing value; and
- degree of contamination.

The bulk density of loose recycled aggregate was determined in accordance with AS 1141.6. As shown in Fig. 1, the density of oven-dried recycled aggregates is generally lower than that of the reference basalt aggregate. The lower density values of recycled aggregates compared to conventional aggregates arise mainly from relatively porous and less dense residual mortar lumps or particles adhering to the surfaces of original natural aggregate particles.

The specific gravity of coarse recycled aggregate ranges from 2.2 to 2.5, which is slightly lower than that of conventional virgin aggregates. The fines fraction of recycled aggregates has a lower density compared to the coarse material.

The absorption capacity of recycled aggregates is variable and generally higher than the standard threshold for primary aggregates. Water absorption values measured in accordance with the procedure described in AS 1141.6 are shown in Fig. 2. The absorption values for coarse recycled aggregate products generally range from 4 to 7% compared with less than 2% for primary aggregates. Higher values up to 8% can be expected for blends of recycled concrete and other materials.

An assessment of the contaminant levels of aggregate batches indicates a limit of no more than 2% inclusive of brick and stony material. Alternatively, the general impurity content comprising friable materials and materials with density less than 1950 $kg/m^3$, is of the order of 1.2% of total material weight, as shown in Fig. 3. Average batch compositions were assessed by visual examination of the +4.75 mm fraction sieved in accordance with AS 1152. Hand sorting of all foreign material was first carried out on the dried material, followed by wetting, to ensure complete identification and classification of all types of contaminants.

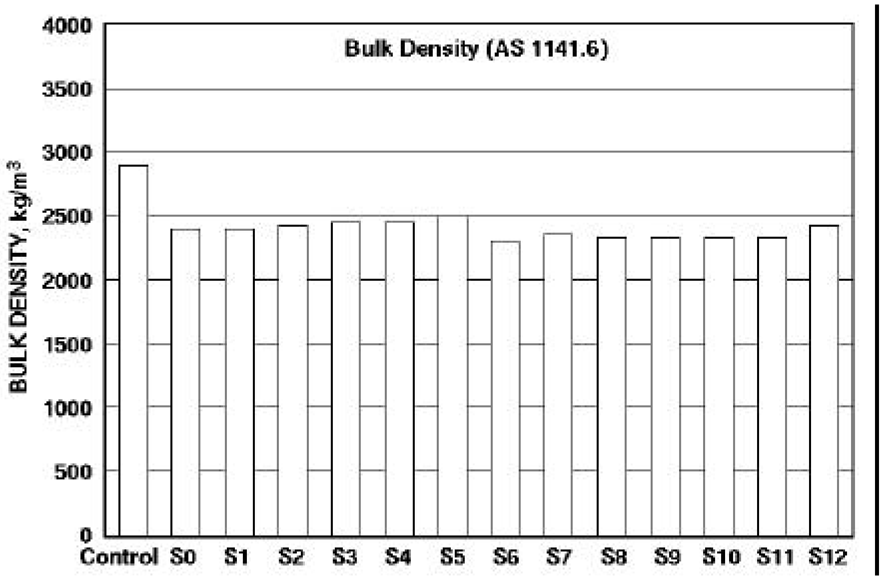


**Figure 1. Plot of bulk density values of aggregates**

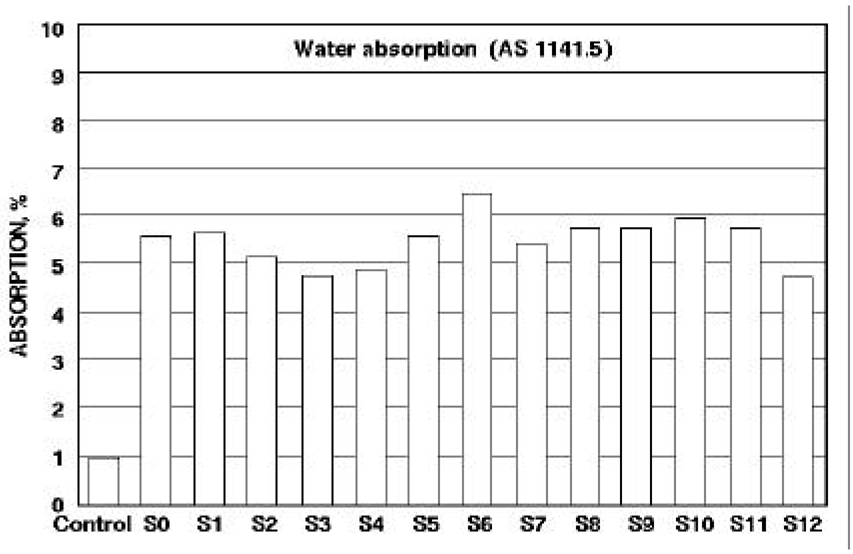


**Figure 2. Water absorption of aggregates**

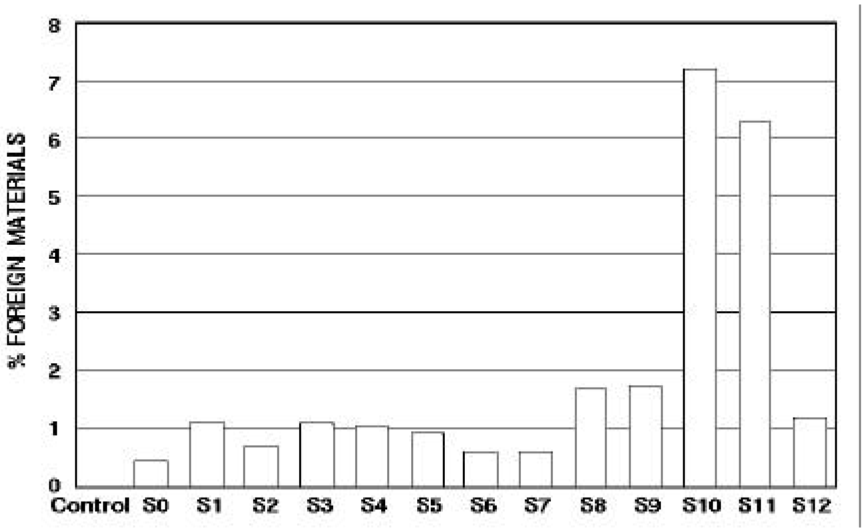


**Figure 3. Total contaminant level expressed as per cent by weight of sample.**

## 4 AGGREGATE FOR CONCRETE PRODUCTION

For concrete production, graded recycled concrete aggregates must be dimensionally stable with regard to variations in moisture content, and sufficiently strong for the desired grade of concrete. The material typically has to comply with grading limits specified in AS 2758.1, and have suitable particle shape and surface texture. Additionally, RCA must neither contain reactive contaminants nor react with cement or reinforcing steel. Table 1 summarises some typical limits in recycled concrete aggregates.

**Table 1. Typical limits recycled concrete aggregates (RCA)**

| *RCA property* | *Class 1A RCA limit* | *Class 1B RCA limit* | *Test method* |
|---|---|---|---|
| Brick content (maximum) | 0.5% | 30% | — |
| Stony material <1950kg/ $m^3$ | 1% | 5% | — |
| Friable material (maximum) | 0.1% | 0.1% | — |
| Particle shape, 2:1 ratio | 35% | 35% | AS 1141.14 |
| Particle density: saturated surface dry (minimum) | 2100 kg/$m^3$ | 1800 kg/$m^3$ | AS 1141.6 |
| Bulk density (minimum) | 1200 kg/$m^3$ | 1000 kg/$m^3$ | AS 1141.4 |
| Water absorption (maximum) | 6% | 8% | AS 1141.6 |
| Aggregate crushing value (maximum) | 30% | 30% | AS 1141.21 |
| Total Impurity level (maximum) | 1% | 2% | |
| LOI (max) | 5% | 5% | |
| Lost substances in washing (maximum) | 1% | 1% | |
| Soundness loss (maximum) | 9% | — | AS 1141.24 |
| Particle size distribution by dry sieving | — | — | AS 1141.11 |

It is recommended that the initial evaluation of RCA for premix concrete production involves direct comparison of the aggregate's physical and mechanical properties, plus the durability characteristics of the concrete, using conventional natural aggregate concrete as the reference material.

## 5 MASONRY RECYCLING

The recycling of building rubble comprising primarily of masonry material presents a much greater challenge for recycling, and severely lags progress in recovery and reuse of recycled concrete. Crushed masonry rubble is a useful material for several different applications in construction, and is mostly used as capping material or as blends with natural aggregates or recycled concrete as drainage or bulk fill material if sufficiently durable. They comprise mainly of burnt clay products, such as bricks, roofing tiles and lightweight blocks. Table 2 shows a summary of quality systems for recycled masonry materials.

Masonry waste often has much higher contamination levels compared to concrete feedstock. The key contaminants are Portland cement mortar, hardboard and plaster. Normal demolition masonry may comprise of components of building materials such as structural lightweight brick; concrete masonry units (block); natural stone; Portland cement mortar, plaster, and terrazzo; gypsum, plaster; ceramic materials; roofing tiles; glass; wood; paper; plastic; asphalt; and metals. The largest source of this class of feedstock arises from residential demolition and renovations. The nature of these contaminants severely restricts possible uses of recycled concrete products, demanding particular effort to separate them at source prior to demolition.

**Table 2. Summary of quality systems for recycled materials**

| *Processing requirements* | *Quality requirement* |
|---|---|
| Feedstock sources | Demolition waste and industrial by-products |
| Feedstock acceptance | Avoidance of contamination by selective demolition and sorting and gate inspection |
| Material storage | Pre- and post-treatment storage, according to product quality and class |
| Material classification | Recycled materials should be classified according to intended use(s) |
| Engineering tests | Engineering tests and frequency of testing should be conducted according to specified standards |

| | |
|---|---|
| Foreign material content | Organic or inorganic contaminants |
| Environmental | Leachability |

The processing of masonry rubble is similar to recycled concrete, but typically generates much lower value-added products. However, there are few premium applications that may be considered for good quality crushed brick rubble such as aggregate for lightweight concrete. Crushed masonry aggregate from various types of demolition debris may also be used in the premix or precast concrete industry.

Contaminants usually are of comparatively less concern in base aggregate applications compared to recycled aggregates which are to be used in new concrete. However, in permeable base applications where the material is not stabilised, dust and fine material wash-offs as well as leachate residues may settle on filter fabric or drainpipes before reaching drainage outlets, clogging pipes, binding filter fabric and ultimately causing system failure. Washing of recycled aggregates can, however, correct this problem.

Typical contaminants in demolition waste feedstock often include lightweight brick and concrete, asphalt, chlorides, cladding, soil and clay balls, steel reinforcement, tiles, vinyl, wood, glass, gypsumboard, hardboard, iron, admixtures and joint sealants, paper, plaster, plastics, rubber and roofing materials of various kinds.

As summarised in Table 3, some of the engineering properties of RCA that are of particular interest when RCA is used as a fill material include grading requirements, strength, specific gravity, durability, and drainage.

**Table 3. Summary of engineering properties**

| | |
|---|---|
| Grading | The crushed and screened material should satisfy specified maximum size and grading requirements for a given application |
| Specific gravity | The specific gravity of RCA aggregates is slightly lower than that of virgin aggregates and ranges from 2.0 for fines to 2.5 for coarse particles |
| Strength characteristics | Processed RCA, being 100% crushed material, is highly angular in shape, and exhibits California Bearing Ratio (CBR) values comparable to crushed limestone aggregates |
| Durability | RCA aggregates generally exhibit good durability with resistance to weathering and erosion |
| Drainage characteristics | RCA is non-plastic; the coarse fraction is free draining and is more permeable than conventional granular material due to lower fines content |

## 6 CONCLUSIONS

The Guide distils existing technical information and potential uses of recycled concrete and masonry waste into a structured format, incorporating specification requirements. It will provide information on material properties and performance. Access to reliable information on all technical material characteristics in a single document will facilitate decisions on product specification, use and marketing.

Through case studies and supplementary field testing, the project will also provide data on the longer term performance of materials derived from recycled concrete and masonry. This durability information will be the first effort in Australia to quantify and report long-term performance. Lack of this data in the marketplace continues to be an impediment to the wider application of recycled resources.

# Quantum Efficiency Of Spectral UV On Polyurethanes

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**Summary: Polyurethanes (PU) are used extensively as exterior building materials. However, PU undergoes degradation during outdoor exposure. The quantum efficiency of spectral UV on photodegradation of an acrylic-urethane coating has been investigated. Samples of approximately 8 mm thick film applied to calcium fluoride substrates were prepared from a mixture of an acrylic polyol with an aliphatic triisocyanate. The UV source was two 1000 W xenon arch lamps, which produced radiation from 270 nm to 800 nm. Ten band pass filters, each having a specified wavelength range, were employed to provide different UV wavelength ranges from 290 nm to 540 nm. In addition, specimens exposed in the absence of UV and under 300 nm cut-on filters were also characterized. Exposure cells containing twelve windows (11 filters and one in the absence of light) corresponding to 12 specimens were used. All specimens were exposed to 50 $^{o}$C and at ~ 0 % relative humidity for more than eight months. The quantum efficiency of each wavelength range was estimated from data on degradation and total dosage absorbed in the film; these properties were measured by Fourier transform infrared spectroscopy and UV-visible spectroscopy, respectively. Both oxidation and chain scission were utilized to quantify PU phodegradation. Although there was no effect of wavelength on the degradation mechanism, both the rate and extent of degradation varied with dosage absorbed at different wavelength ranges. Except for the 290 nm filter, which had the highest value, quantum efficiency of other wavelength ranges did not differ greatly.**

**Keywords: Polyurethane, photodegradation, spectral UV, wavelength. quantum efficiency**

## 1 INTRODUCTION

Polyurethanes (PU) are used extensively as exterior coatings, sealants, and adhesives. However, PU materials undergo degradation during exposure to outdoor environments (Bauer et al. 1991, Wernstahl 1996). Although temperature, moisture, and other weathering elements may contribute to the degradation, the primary cause of PU degradation during outdoor exposure is ultraviolet (UV) light. For that reason, the photodegradation mechanism of PU and its model compounds have been studied extensively (Wilhelm and Gardette 1997 and 1998, Rabeck 1995). Both aromatic and aliphatic isocyanate-based polyurethanes undergo chain scission by photo-oxidation process. It is well known that photodegradation (photolysis and photo-oxidation) of a material is determined by the absorption characteristics of radiation in that material. However, the absorption of radiation energy in polymers is controlled by their chemical structure and is dependent on wavelength. Studies on wavelength sensitivities of a variety of polymers have been summarized (Searle 1986, Trubiroha 1986), and some data on the effects of wavelength on the PU degradation characteristics have been documented (Gardette and Lemaire 1984, Wilhelm and Gardette 1997). However, very little research on the quantum efficiency of spectral UV on PU has been reported (Gardette and Lemaire 1984). This is the subject of the study reported in this paper.

The quantum efficiency is estimated by:

$$N(\lambda) = D_{dam}(\lambda, t) / D_{dos}(\lambda, t)$$

where

$\phi(\lambda)$ = apparent quantum efficiency within the exposed radiation wavelengths, in $Am^{-1}J^{-1}$ (A is infrared absorbance and m is film thickness),$D_{dam}$ = damage, in IR absorbance (A) units.

$D_{dos}(\lambda, t)$ = total dosage absorbed in the material, in J.

The total absorbed dosage is the total number of quanta absorbed by a material and is given by

$$D_{dos}(t) = \int_0^t \int_{\lambda \min}^{\lambda \max} E_o(\lambda)\left(1-10^{A(\lambda)}\right) d\lambda \ \mathrm{dt}$$

where

$\lambda_{min}$ and $\lambda_{max}$ : minimum and maximum photolytically effective UV-visible wavelengths (units: nm),

A(λ,t) : absorbance of sample at specified UV wavelength and at time t, (units: dimensionless),

$E_o(\lambda,t)$ : incident spectral UV-visible radiation dose on sample surface at time t (units: J $cm^{-2}$),

T : elapsed time (units: s).

The total absorbed dosage was obtained by integrating the product of the spectral irradiance, $E_c$ (8,t), and the spectral absorption of the coating, $(1\text{-}10^{-A\,(8,\,t)})$, over the wavelengths of radiation impinging on the specimen for the exposure duration at a particular humidity and temperature. Both quantities were measured using UV-visible spectroscopy and damage was measured with FTIR. The quantum efficiency determined in this study is considered as an "apparent" value and is expressed as the change in FTIR intensity per unit thickness per amount of radiation absorbed in the coatings. If FTIR intensity of the degradation is expressed in absorbance units (A), thickness in meters (m), and radiation dosage absorbed in Joules (J), the apparent quantum efficiency is expressed in A $m^{-1}$ $J^{-1}$. Since both the absorbed dosage and material damage were measured on exposed specimens, the quantum efficiency does account for the effects of relative humidity, temperature, and radiant flux on the degradation.

## 2 EXPERIMENTAL PROCEDURES

### 2.1 Material

A model thermoset acrylic-urethane coating consisting of a mixture of a hydroxy-terminated acrylic resin and an aliphatic isocyanate cross-linking agent was used. The acrylic resin contained, by mass, 68 % butylmethacrylate, 30 % hydroxy-ethylacrylate, and 2 % acrylic acid. The isocyanate was a conventional biuret hexamethylene diisocyanate. The urethane was formulated without catalyst and at an OH - NCO ratio of 1. Coatings were applied to calcium fluoride ($CaF_2$) substrate by spin coating followed by curing at 130 °C for 20 minutes. All coated samples were well cured, which was evidenced from Fourier transform infrared spectroscopy (FTIR) analysis of the NCO band at 2272 $cm^{-1}$. $CaF_2$ is transparent in the wavelength range between 0.13 μm and 11.5 μm and has excellent moisture and heat resistance. Therefore, it was possible to expose clear coated $CaF_2$ to wide ranges of temperature, humidity, and UV radiation, while monitoring the spectral absorption and degradation of the coating by UV-visible spectroscopy and FTIR spectroscopy. The coating thickness was 8 :m ∀ 1 :m.

Instrumentation and Exposure Cells

Figure 1 schematically shows the main components of the experimental setup used in this study. It consists of two solar simulators (the light source), an exposure cell platform, a controlling system, and instruments to measure spectral radiation energy and coating degradation. Each solar simulator's optical system is comprised of a 1000 W xenon arc lamp, a dichroic mirror, an optical integrator, and a Fresnel collimating lens. The dichroic mirror removes the visible and infrared portions of the xenon arc spectrum; therefore, the thermal effect induced by the radiation on the specimens was less than 2 °C above ambient temperature. The integrator homogenizes the beam while the Fresnel len collimates the beam.

The exposure cells were designed to simultaneously expose different sections of the same film to 12 well-defined, spectral UV conditions at the same temperature and relative humidity (RH). Figure 2a depicts the arrangement of the exposure cells underneath the solar simulators, and Figure 2b shows a cross section of the exposure cell. Each exposure cell has a layered design and included a filter disk, a quartz disk, a $CaF_2$ specimen disk, and an encasement to hold the disks. The filter disk contained 11 windows--10 on the disk perimeter and the eleventh in its center. Each window was 16 mm in diameter. Each of the 10-perimeter windows was outfitted with a band pass filter covering a different segment of xenon arc spectral spectrum from 290 nm to 540 nm. The center window was outfitted with a 300 nm cut-on filter (allowed only wavelengths >300 nm to be transmitted). The broadness of each wavelength range and the transmission of the 10 band pass filters are given in Figure 3. The first eight filters had full-width-half-maximum (FWHM) values between 2 nm and 10 nm and covered the range between 290 nm and 340 nm. The nominal center wavelengths for these filters were 290 nm, 294 nm, 300 nm, 306 nm, 312 nm, 318 nm, 326 nm, and 336 nm. The two remaining filters had FWHM values greater than 10 nm and had center wavelengths of 354 nm and 450 nm. Hereafter, the band pass filters are also referred to by their center wavelengths, e.g., 290 nm filter, 354 nm filter, etc. In addition, changes in specimens exposed in the absence of UV (no-UV) (no. 12 in Figure 2a) were also measured to assess non-photolytically induced degradation.

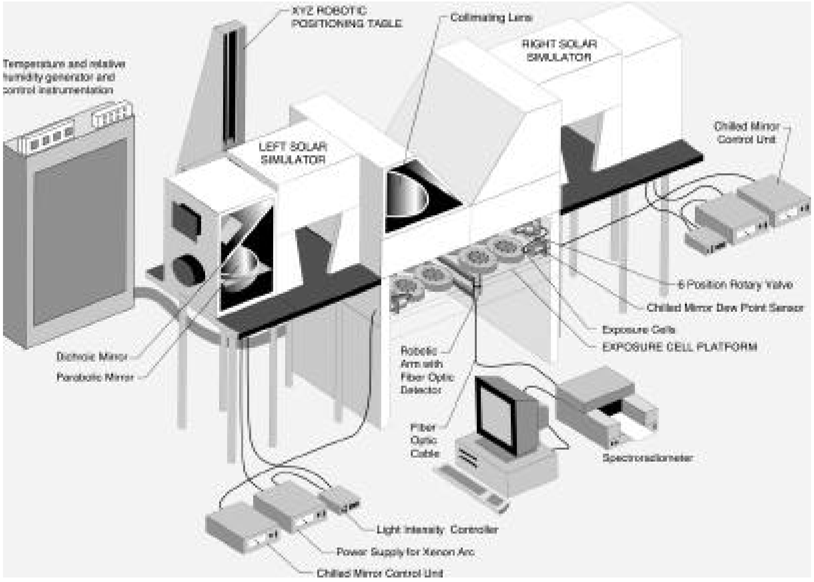


**Figure 1. Experimental setup for measuring absorbed dosage and quantum efficiency of coatings exposed to different spectral UV/relative humidity/temperature conditions.**

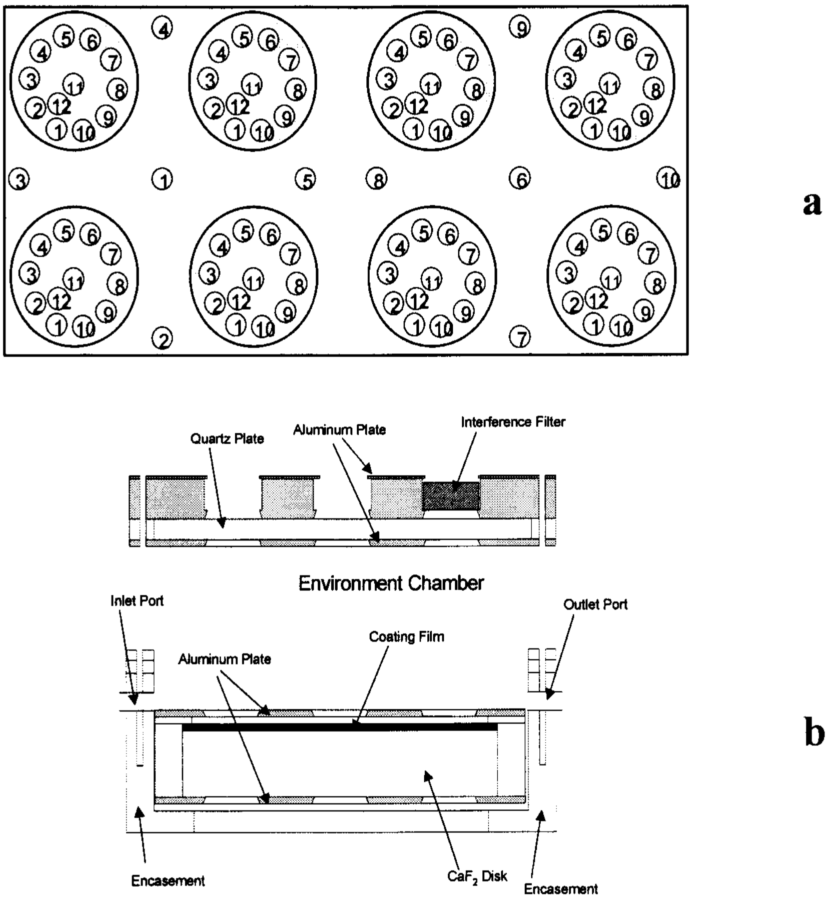


**Figure 2. a) Arrangement of exposure cells, and b) cross section of an exposure cell.**

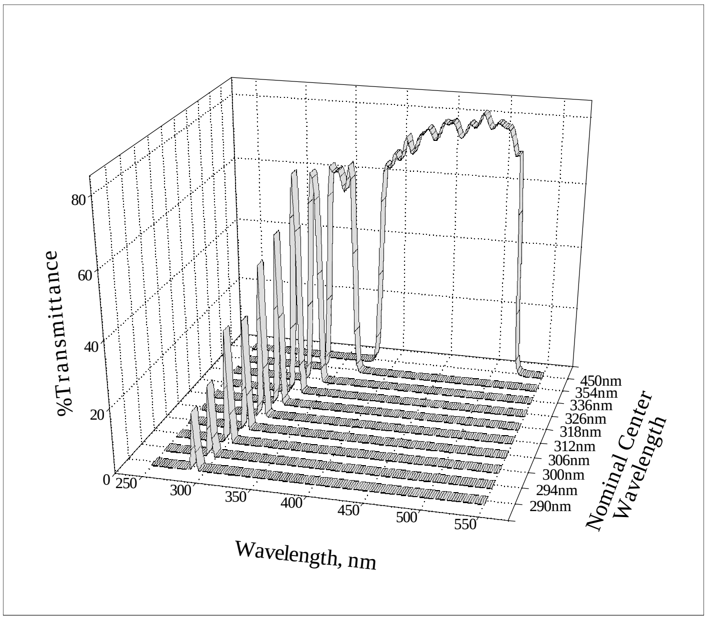


**Figure 3. Characteristics of the 10 band pass filters used to provide different spectral UV ranges.**

The space between the quartz disk and coated $CaF_2$ disk served as the environmental exposure chamber. The exposure cell was machined to tight tolerances so that a positive pressure could be maintained within the exposure chamber of an exposure cell. Inlet and outlet ports were drilled into the exposure cell to circulate the desired temperature/RH condition into the chamber. The conditioned air was generated in a humidity-temperature generator and pumped into each exposure cell. The humidity generators were fabricated based on the principle of dry/moisture-saturated air mixture. The temperature within the exposure chamber was monitored via a thermocouple and RH was monitored by a chilled mirror hygrometer. The RH at a specified temperature in each cell was tracked by a feedback-control system, which monitored both the temperature and RH within the exposure cell three times per second. The temperature and RH could be maintained within 0.5 % and 3 %, respectively, of their set values.

## 2.2 Measurements of Absorbed Dosage and Coating Degradation

To accurately estimate the total dosage absorbed in the film, the optical properties of all of the components must be measured, including 1) the spatial and spectral variability in the radiant flux of the light source, and 2) spectral transmittances of the band pass filters, quartz disks, and coated $CaF_2$ disks for all eight exposure cells. Spatial and temporal changes of lamp intensity were monitored using an UV-visible spectrometer connected to a robotic controlling system via a fiber optic cable. These measurements were done in the absence of the exposure cells. Measurements of the optical properties of the system components were highly automated by the use of a robotic arm and an automated sampler. The robotic-controlled fiber optic cable probe had the capability of moving in the X, Y, and Z directions, and the spectrometer was programmed to make measurements at frequent time intervals at 98 different locations on the platform table (88 corresponding to 11 filters on each of the eight exposure cells and 10 locations outside the exposure cells).

UV absorption in the coatings was obtained from the transmission of light passed through the band pass filters, the quartz plate, and the coated $CaF_2$ plate. At each specified time interval, the coated $CaF_2$-contained exposure cell was removed from the platform table and fitted into a demountable 150-mm diameter ring of an auto-sampling accessory. The ring was computer-controlled and can be programmed to rotate and translate over the entire sampling area. Spring-loaded Delrin clips ensure that the specimens are precisely located and correctly registered. The auto-sampler was placed in the UV spectrometer compartment and UV-visible spectra were taken at 2 s per scan using a customized computer program. This automated sampling system allowed unattended, efficient, and quick recording of the UV transmission of the coating at all 12 windows of an exposure cell. Since the exposure cell was mounted precisely on the auto-sampler, error due to variation of sampling at different exposure times was essentially eliminated. Transmission measurements were also taken individually for the quartz, the coated $CaF_2$, and the bare (uncoated) $CaF_2$ plates. To improve the accuracy of these measurements, the UV-visible spectrometer was calibrated during each inspection against standard UV-visible spectrometer transmittance density filters using NIST Standard Reference Material (SRM 930d). From the amount of light transmitted through each component of the exposure cell as a function of time, the light intensity impinging upon, as well as absorbed by, the coatings was determined. Both of these quantities are required inputs for estimating the absorbed dosage and quantum efficiency.

Damage in the coating was measured by transmission FTIR spectroscopy. FTIR spectra were taken using the same auto-sampler described above placed in an FTIR spectrometer equipped with a liquid nitrogen-cooled mercury cadmium telluride (MCT) detector. All FTIR spectra were the average of 132 scans recorded at a resolution of 4 $cm^{-1}$ using dry air as the purge gas. The peak height method was used to represent IR intensity, which is expressed in absorbance, A. The FTIR spectrometer was calibrated against a polystyrene film during each inspection. All results were the average of three specimens. A software program was written to estimate total absorbed dosage, material damage, and quantum efficiency.

## 3 RESULTS AND DISCUSSION

### 3.1 Spatial Uniformity, Collimation, and Temporal Stability of UV Light Source

Results on spatial uniformity, collimation and temporal stability of the UV light source have been given elsewhere (Martin et al, 2001). In general, the spectral radiant flux of the xenon arc light sources changed with time. The magnitude of the changes differed from one wavelength to another. For example, after 1500 h of operation, the spectral radiant flux decreased, with respect to initial values, at 290 nm, 326 nm, and 450 nm by 25 %, 50 %, and 38 %, respectively. These changes demonstrate the need for frequent monitoring of the spectral radiant flux of the light source. The radiant flux also differed from one solar simulator to another and varied from one lamp to another. Moreover, the spatial variation in the radiant flux over the working plane of both simulators was high. This high spatial variability in the irradiance pattern necessitates a large number of measurements to characterize the spatial variation.

Beam collimation and scattering of the radiation were ascertained by measuring the radiant flux at the center points of several randomly selected windows. Measurements started at a point immediately below the exposure cell window and, then, at 10 mm increments below this point until the sensor was 100 mm vertically beneath its initial position. Radiant flux measurements were plotted against vertical distance and regression analysis was performed. The slope of the regression curve was not significantly different from zero suggesting that the radiant output from each solar simulator was highly collimated. The absence of stray light incident on the covered 12$^{th}$ window was verified by positioning the fiber optic probe immediately beneath this window; no UV-visible radiation was detected. The band pass filters were photolytically and thermally stable, except for the 450 nm filter, which lost 2 % to 7 % of its transmittance after over 1000 h exposure. The $CaF_2$ did not exhibit any transmittance change and the quartz plates showed a small transmission loss over the course of exposure. All of these changes were accounted for in the calculation of absorbed dosage.

### 3.2 Coating Degradation

Examples of FTIR results on coating degradation are given in this section to demonstrate the approach for measuring the material damage and calculating the spectral quantum efficiency in the samples. Figure 4 presents typical FTIR spectra of the coating before and after aging for two different times under the 300 nm cut-on filter (KG) at 50 $^{o}$C/~0 % RH condition. The major FTIR bands of interest in a cured, unaged acrylic-urethane coating based on aliphatic isocyanate are the bands at 3380 cm$^{-1}$ due to NH stretching, 1730 cm$^{-1}$, due to C=O of urethane linkage, 1520 cm$^{-1}$, due to amide II (NH bending and CN stretching) and 1250 cm$^{-1}$, due to amide III (NH bending and CN stretching). The assignments are based on extensive IR data of polyurethanes (Scrichatrapimuk and Cooper 1978).

Although a comparison between spectra of the unaged and aged samples, as shown in Figure 4, can give some information about the effects of exposure on a polymer film, the degradation behavior is best studied from the difference spectra, where the formation or depletion of a band and the appearance of a new species can be readily observed. Examples for such difference spectra as a function of spectral UV ranges and exposure times for samples exposed under the cut-on filter (KG) in 50 $^{o}$C/~0%RH condition are displayed in Figures 5a and 5b, respectively. The bands above and below the zero line in a difference spectrum indicate a gain and loss, respectively, of chemical species in the film.

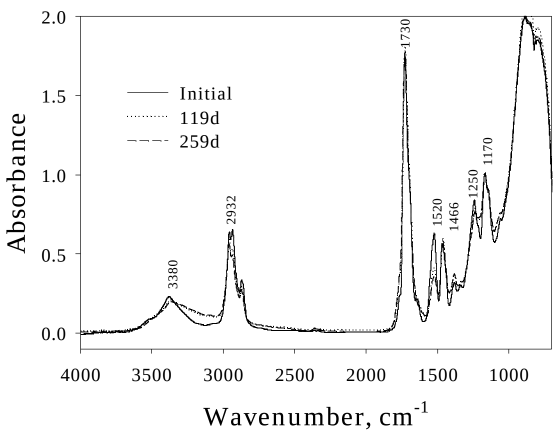


**Figure 4. Typical FTIR spectra of PU coating before and after exposures for two different times under the 300 nm cut-on filter in 50 $^{o}$C/~0% relative humidity condition.**

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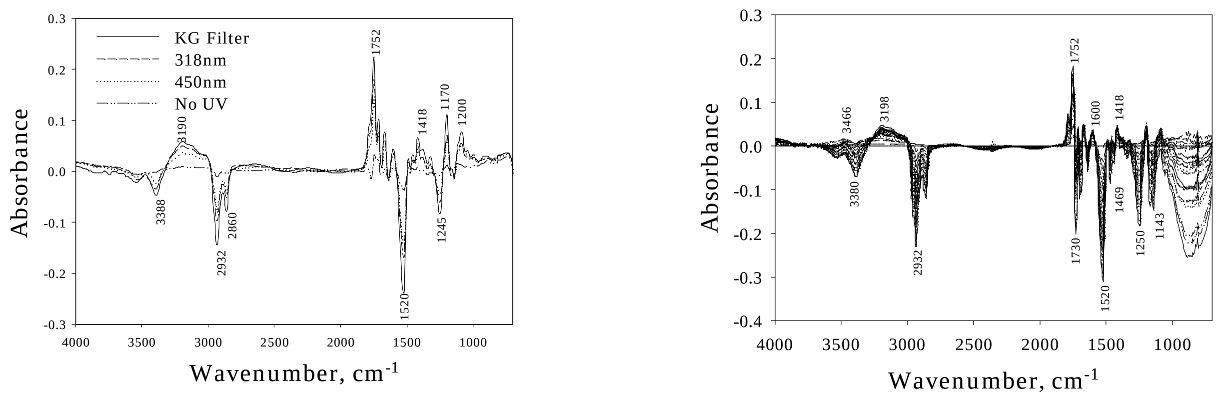
The degradation observed at 50 °C/dry condition in the absence of UV light (no-UV) was small compared to that of other UV light ranges. Various compounds were formed during exposure to UV, as evidenced by the appearance of the bands at 1752 $cm^{-1}$, 1418 $cm^{-1}$, and in the 3100 $cm^{-1}$ to 3300 $cm^{-1}$ region. Similarly, the intensity of the bands at 1730 $cm^{-1}$, 1520 $cm^{-1}$, 1250 $cm^{-1}$ and in the CH region between 2800 $cm^{-1}$ to 3100 $cm^{-1}$ decreased markedly with exposure times. These results indicate that the PU material has degraded substantially after more than eight months of exposure. Further, all the bands that were formed occurred at the same positions irrespectively of the wavelength ranges studied. The same band positions in the IR spectra indicate that the same chemical groups were produced during the photodegradation at different wavelengths. This result suggests that, despite a substantial variation in the extent of degradation among the UV wavelength ranges, essentially no effect of wavelengths on the primary photodegradation mechanism of this polyurethane material was observed. Such wavelength insensitivity implies that the degradation in this material was induced by chromophores initially present in the films.

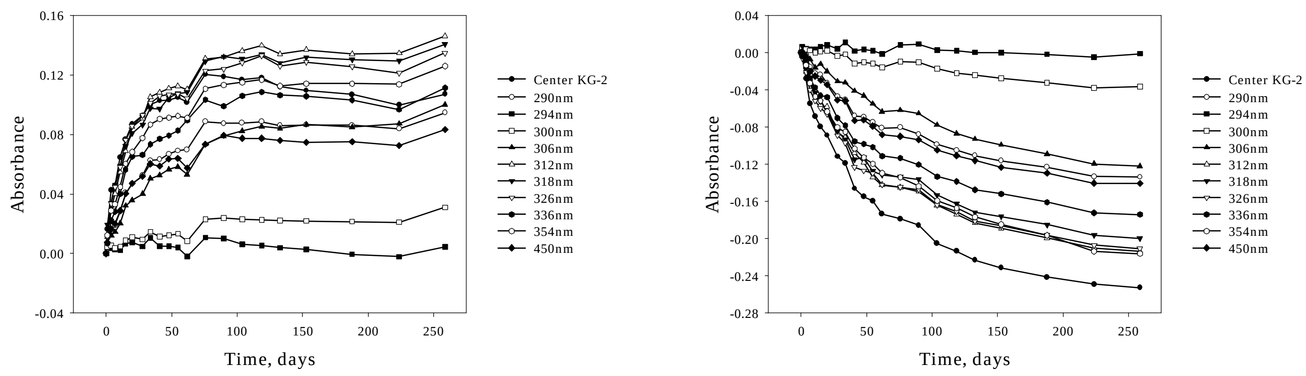
The C=O band at 1752 $cm^{-1}$ (from acetyl urethane, O=C-NH-C=O ) and the NH band at 1520 $cm^{-1}$ were used to follow the oxidation and chain scission, respectively. Figures 6a and 6b depict FTIR intensity changes of these two bands as a function of time for the cut-on filter and 10 center wavelengths.  In these figures, each symbol represents a data point, which was the average of three specimens, as stated  in the experimental section. The coefficients of variation for all data points were <7 %, indicating that the reproducibility of the results between the specimens was good. As can be seen in these figures, there is no particular order for both the rate and magnitude of degradation with regards to wavelength despite the fact that the shorter wavelengths contain higher-energy photons. However, the 294 nm and 300 nm filters appeared to cause the least damage. This was true for both oxidation and chain scission. The oxidation for all wavelength ranges was greater at earlier time then became almost constant at prolonged exposure. For chain scission, the degradation was fast during the first 50 days but slowed down thereafter.




a

b

**Figure 5. Difference spectra of PU exposed for 180 d under no UV and several spectral UV ranges (a), and for different times under the 300 nm cut-on filter (b), in 50 °C/~0 % RH condition.**



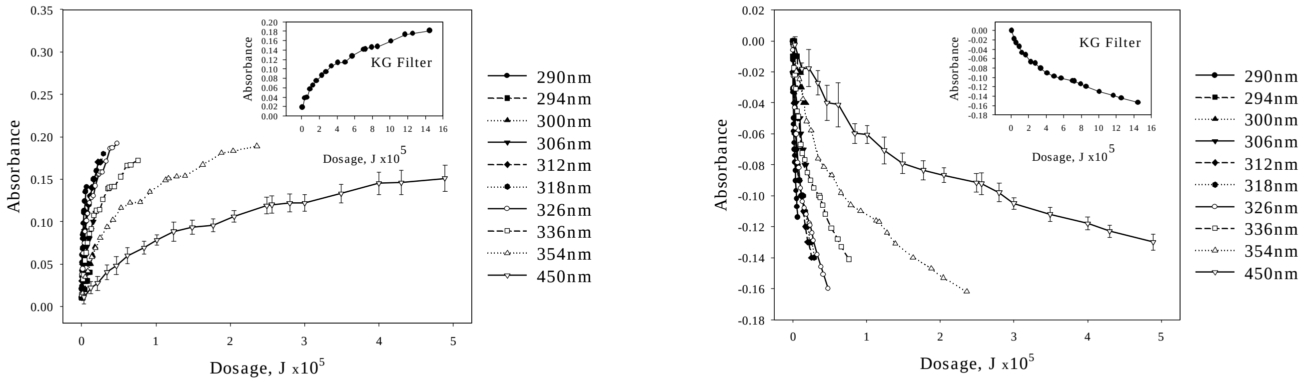

a

b

**Figure 6. FTIR intensity changes of PU with exposure time in 50 °C/~ 0%RH for the 300 nm cut-on filter (KG) and 10 different center wavelengths; a) oxidation, and b) chain scission.**

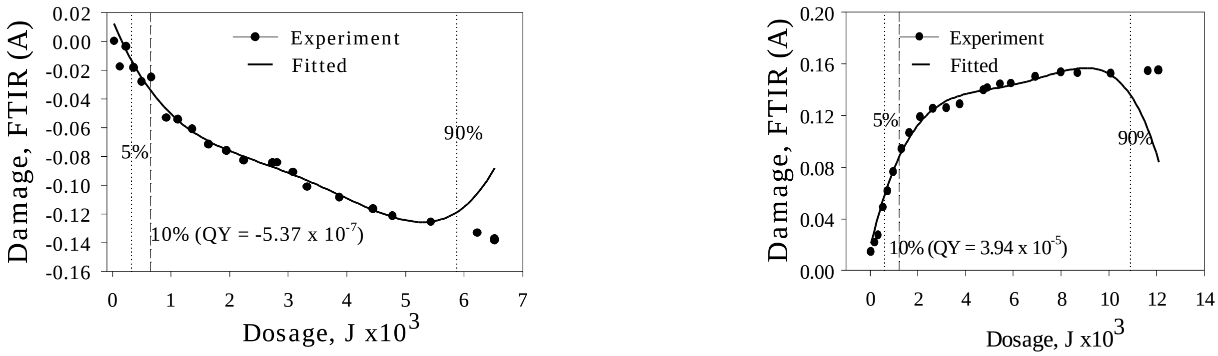
### 3.3 Total Absorbed Dosage and Apparent Quantum Efficiency

Photons of lower wavelength radiation are more energetic than those of higher wavelengths, and far fewer photons of the lower wavelength radiation actually impinged upon the sample surface. Therefore, the damage/time relationship as shown in Figure 6 is not valid for comparing the effects of different wavelengths on polymer degradation. The relative effects may be observed in the damage/absorbed dosage plot because only radiation absorbed in the material that causes degradation is really taken into account. Figures 7a and 7b display photo-oxidation and chain scission, respectively, as a function of absorbed dosage for specimens exposed to cut-on filter and 10 band pass filters under 50 °C/~0 %RH condition. It should be noted that only data involved in the photolytic process are included in Figure 7. That is, the results measured in the absence of UV light at the same temperature and RH were removed from the total degradation. The absorbed dosage, i.e., the actual amount of radiation absorbed by the coating, was calculated using the equation given above. All dosages were measured after 250 d of exposure. As expected, samples under the cut-on filter (KG) and long wavelength band pass filters absorbed the most dosage while those exposed to short wavelengths absorbed less. Further, radiation from the lower wavelength filters caused more rapid degradation than higher wavelengths, as evidenced by the slopes of the curves.




**a** **b**

**Figure 7. FTIR intensity change with absorbed dosage for PU exposed to 50 °C/~ 0%RH condition for one cut-on filter (KG) and 10 different center wavelengths; a) oxidation, b) chain scission.**




**a** **b**

**Figure 8. Experimental and fitting FTIR degradation data as a function of dosage for a) chain scission, and b) oxidation. The initial slopes of these curves are used for obtaining the quantum efficiency.**

The apparent quantum efficiency was taken as the initial slope of the damage/total absorbed dosage curve. The use of this initial slope avoids complications from multiple degradation processes and from potential shielding of the coating material by degradation products absorbing radiation. In the process of determining this slope, a fourth-order polynomial function was fitted to the damage/total absorbed dosage curve. In this first implementation of analyzing this curve, the slope at the 10 % of the total absorbed dosage was used as the end point. One example of such fitting is shown in Figure 8 for the chain scission process (1520 cm$^{-1}$ band) and oxidation (1752 cm$^{-1}$ band) of samples under the 450 nm filter. In this figure, the symbols are the experimental data and the solid lines are the fitted curves. The initial part of the damage/absorbed dosage curves at 1520 cm$^{-1}$ exhibits some scatter due to imprecision in the measurements of very small changes at early exposures. Nevertheless, the polynomial function fit most of the data well. Differentiating the polynomial gives the slope (apparent quantum efficiency) at a given absorbed dosage. Since the damage/absorbed dosage relationship is not linear, the apparent quantum efficiency falls off with total absorbed dosage, presumably due to increasing shielding of the coating by degradation products and to the depletion of weak links in the material. Note that the negative values in Figure 8a vertical axis are indicative of a loss of material.

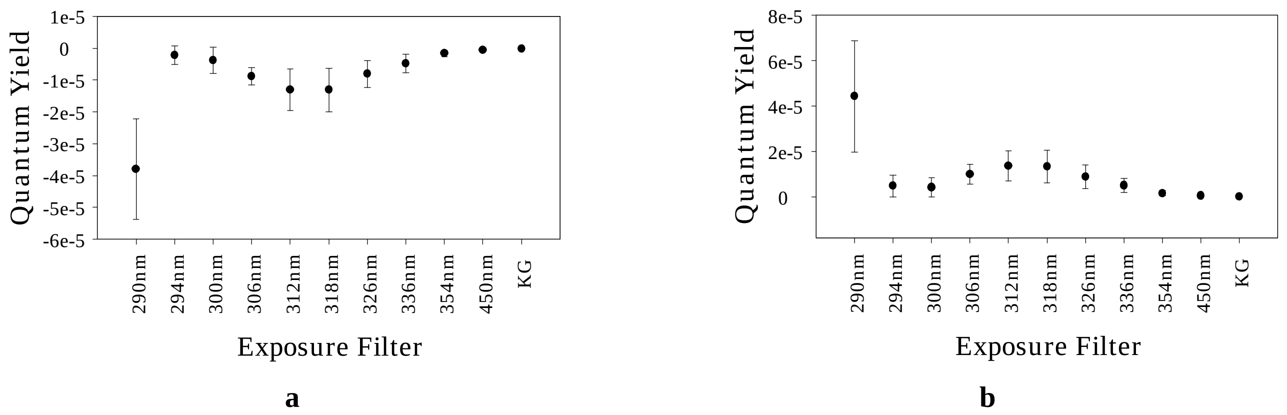



**Figure 9. Quantum efficiency of a) chain scission and b) oxidation of an acrylic-urethane coating exposed to 50 °C/~0 % RH for 300 nm cut-on filter and 10 different center wavelengths.**

Figure 9 presents apparent spectral quantum efficiency for the oxidation and chain scission of PU coating exposed to 50 ° C/~0 %RH for a cut-on filter (KG) and ten different center wavelengths. The one-standard error bars are included in the curves. The quantum efficiency at 290 nm appears to be the highest. Although the samples under the cut-on filter and long center wavelengths, i.e., 450 nm and 354 nm filters, absorbed the most energy, their quantum efficiency was similar to other filters. Since the unit employed to express quantum efficiency in this study is different from that traditionally used (number of molecules that undergo change/number of quanta absorbed), it is difficult to compare directly the values obtained here with data reported for other polymers. Work is in progress to measure the molar absorption coefficients of the infrared bands of interest, and the results will be used to convert IR absorbance units into number of molecules. Nevertheless, the quantum efficiencies in all spectral UV ranges for this PU were small, as is typical for solid polymers. For example, the quantum yield of hydroxyl formation in polyurethanes exposed to 315 nm and 365 nm radiation has been reported to be $0.002 \pm 0.001$ and $0.005 \pm 0.001$, respectively (number of OH formed/number of quanta absorbed) (Gardette and Lemaire 1984). Quantum yields of chain scission for other polymers also range between $10^{-2}$ to $10^{-5}$ (Randy and Rabeck 1975). It should be mentioned that the quantum yield of polymer chain scission changes rapidly from below to above the glass transition temperature (Tg). Above Tg, the quantum yield is similar to that irradiated in solution, which is several order of magnitude greater than that in solid (Randy and Rabeck 1975). Since water in the films tends to plasticize and decrease the Tg of polymers, photodegradation of PU materials is expected to be more rapid if they are used at temperatures higher than their Tg.

Total absorbed dosage and apparent quantum yield have been found to be very sensitive to a number of experimental variables, including the initial UV-visible absorbance of the coatings, formation of coating degradation products, and UV-visible and FTIR measurement errors (Martin et al. 2001). As a result of these studies, protocols have been developed and experimental errors had been essentially eliminated through careful and frequent re-calibration of the UV-visible spectrophotometer using NIST standard UV-visible spectrometer transmittance density filters and polystyrene films to monitor the performance of the FTIR spectrometer before taking the spectra of any samples.

## 4 SUMMARY AND CONCLUSIONS

Polyurethanes are used extensively as exterior building materials. However, these materials undergo degradation during outdoor exposure. The quantum efficiency of spectral UV on photodegradation of an acrylic-urethane coating was investigated. Damage was measured by transmission Fourier transform infrared spectroscopy, and energy absorbed in the sample for each wavelength range was estimated from measurements of spectral irradiance and spectral absorption of the coating using UV-visible spectroscopy. Ten spectral wavelength ranges covering from 290 nm to 540 nm were generated using band pass filters, which allow certain radiation wavelengths passing through and interact with the samples. Samples exposed in the absence of UV and under 300 nm cut-on filters were also analyzed. The UV light source was provided by two 1000 W xenon arch lamps, which contained radiation from 270 nm to 800 nm. Samples were prepared by applying a mixture of an acrylic polyol with an aliphatic triisocyanate to calcium fluoride substrates. They were exposed to 50 °C and ~ 0 % relative humidity for more than eight months. FTIR results showed that the degradation mechanism of PU was essentially the same regardless of the center wavelengths from 290 nm to 450 nm. Both the rate and extent of the photodegradation with respect to absorbed dosage varied with wavelength. However, except for the 290 nm filter, which appeared to have the highest value, the quantum efficiency of other wavelength ranges did not differ greatly. Knowledge of the spectral quantum efficiency in the UV-visible region is essential for developing improved photo-accelerated tests and more photo-stable materials.

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## 5 ACKNOWLEDGEMENTS

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# Surface And Interface Microstructures Of Coatings Systems And Their Implication On Service Life

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**Summary: The surface and interface properties have a strong influence on the adhesion and service life of polymer coatings applied to plastics and metals. However, both the surface and interface chemical composition and microstructure of coated materials are affected by the surrounding environments and processing conditions during film formation. In this study, the surface and interface properties of several coatings have been characterized with nanoscale resolution using atomic force microscopy (AFM). Specimens were prepared by applying thick films of coatings on the substrates. Both the cross-sections and surfaces that were in contact with the substrates during curing were used for the interface characterization. Interpretation of AFM results was aided by data collected using attenuated total reflection-Fourier transform infrared spectroscopy and contact angle measurements. Curing conditions and stoichiometry have a strong influence on the interface region. Further, microstructure and chemical composition of the film surface adjacent to the substrate are different from those of the surface exposed to air. The implications of these differences on the wettability, weatherability, and adhesion of coated systems are discussed.**

**Keywords: Interface, surface, coatings, nanoscale, AFM**

## 1 INTRODUCTION

The surface and interface properties have a strong influence on the service life and adhesion of coatings applied to plastics and metals. However, both the surface and interface chemical composition and microstructure of coated materials are affected by the environment and processing conditions during film formation. For example, it has been established that the air surface of a multicomponent polymer system is generally enriched by a thin layer of hydrophobic material. Further, water, $CO_2$, and oxygen from the environment can react with certain components of a coating formulation to form side products on a coating surface. For example, water reacts readily with isocyanate in polyurethane formulations to form urea; $CO_2$ form salts with amine curing agents in amine-cured epoxies; and oxygen inhibits the curing of acrylic resins. Such surface enrichment and environment-induced reactions would generate a surface microstructure and chemical composition that are different from those in the bulk or at the film/substrate interface. Similarly, the buried interface/interphase between a coating and a substrate is often affected by processing conditions that control the chemical kinetics, diffusion, and volumetric changes. In addition, the interface region often includes impurities, unreacted molecules, and additives. The resulting interface/interphase is often a very complex structure, which is not easily analyzed or modeled. In this study, the interface is defined as the two-dimensional boundary between the polymer film and the substrate, while the interphase is the three-dimensional region including the interface and a zone of finite thickness on both sides of the interface. Considerable research has been conducted to improve the strength and service life of the interface/interphase region through surface treatment or modification of the substrates. However, little progress has been made in the understanding of the morphological, physical, chemical, and mechanical properties of this region, how these properties change with external stresses and processing conditions, and how this region affects the performance and service life of a polymeric system.

The main objective of this study is to characterize the surface and interface morphologies of polymer- coated materials by the use of nanoscale-resolution tapping mode atomic force microscopy. The results are presented to demonstrate that a substantial difference exists between the microstructure of the film surface exposed to air and the film surface adjacent to the substrate. These differences may have strong implications in the wettability, adhesion, and weathering resistance of coatings systems.

## 2 MATERIALS AND INSTRUMENTATION

### 2.1 Materials

Surfaces of polymer films exposed to air (air surface) and the surfaces of the same films that were in contact with the substrate during film formation (interface) were characterized using tapping mode AFM (TMAFM). In addition, the interface/interphase region of cross-sectioned samples was also studied. Samples of several different coatings having thicknesses between 50 µm and 200 µm applied to substrates were prepared by a draw-down technique. Highly-polished silicon wafers and glass plates were employed as the substrates to provide the interface samples. They were obtained by first initiating a cut through the cured coated film with a razor blade, followed by lifting up the film with a sharp knife, and peeling with a tweezers. The cross-sectioned samples were prepared from coatings applied to polished steel. Strips of coated steel were imbedded in a molding compound. Cross sections were obtained by cutting the embedded samples with a diamond saw, followed with several polishing steps; the final polish was with a 0.25 µm diamond paste.

### 2.2 AFM measurement

Although TMAFM is a recently-developed technique, it has become a popular tool for studying surface microstructure of polymers and biological materials because it provides a high lateral resolution with minimal damage to the samples (Magonov, 2000). In tapping mode (Fig. 1), the cantilever carrying the probing tip is oscillated at a frequency near its resonance frequency, typically, a few hundred kHz, so that the tip makes contact with the sample only for a short duration in each oscillation cycle. As the tip approaches the sample, the tip-sample interactions cause a change in the amplitude, resonance frequency, and phase angle of the oscillating cantilever. Detection of changes in the amplitude and phase shift of the cantilever probe during scanning provide topographic and phase images, respectively.

All AFM measurements were performed using a Dimension 3100 AFM (Digital Instruments)** under ambient conditions. Both AFM topographic (height) and phase images were taken. For most of the images, a set point amplitude of between 50 % and 70 % of the free amplitude was used. Silicon tips having a drive frequency of about 300 kHz and a radius of approximately 5 nm were used.

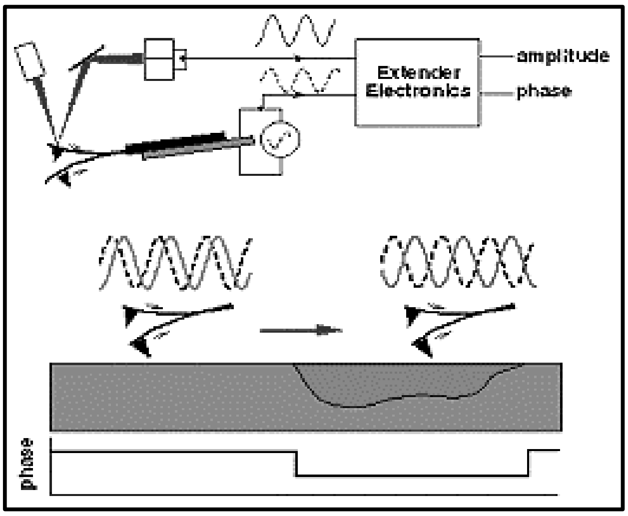


**Figure 1. Tapping Mode AFM, showing cantilever oscillation amplitude and phase shift during scanning.**

## 3 RESULTS AND DISCUSSION

### 3.1 AFM images of air surface and film/substrate interface

Samples of two commonly-used coatings, an amine-cured epoxy and an acrylic-melamine, applied to a very smooth silicon wafer were used to examine morphological differences between the air surface and film/substrate interface of coating systems. AFM images of the surface (a) and interface (b) for the amine-cured epoxy are displayed in Figure 2. The material was a stoichiometric mixture of a bisphenol A epichlorohydrin epoxy resin having an epoxide equivalent mass of 3050 g/mol and a polyalkoxyamine curing agent. Solutions of 50 % (by mass) epoxy resin in toluene and 50 % (by mass) amine curing agent in toluene were thoroughly mixed at appropriate ratios and applied to the Si substrates. The substrates were cleaned with acetone, followed with methanol, and dried with hot air. Coated samples were cured for 24 h at ambient condition (24 °C and 45 % RH), followed by post curing at 120 °C for 2 h. The fully-cured film had a glass transition temperature of approximately 98 °C, as measured by differential scanning calorimetry. For both Figs. 2a and 2b, the pictures on the left, center, and right correspond to 3-D (dimension) topographic, 2-D topographic, and phase images, respectively. The bright and dark areas in the topographic images correspond to the surface peaks and valleys, respectively. It should be mentioned that both topographic and phase AFM

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imagery taken at high magnifications showed that the silicon surface is very smooth and essentially featureless with no evidence of a surface pattern.

Figure 2 shows several morphological differences between the surface and the interface of amine-cured coatings. First, the face in contact with the substrate during film formation is considerably rougher than that of the surface exposed to air, as evidenced by the 3-D images of Fig. 2. The average roughness, root mean square (RMS), of the surface is approximately 3 nm as compared to 10 nm for the interface. It is not known whether the rough interface was actually formed during the film formation or was caused by peeling the film from the substrate.

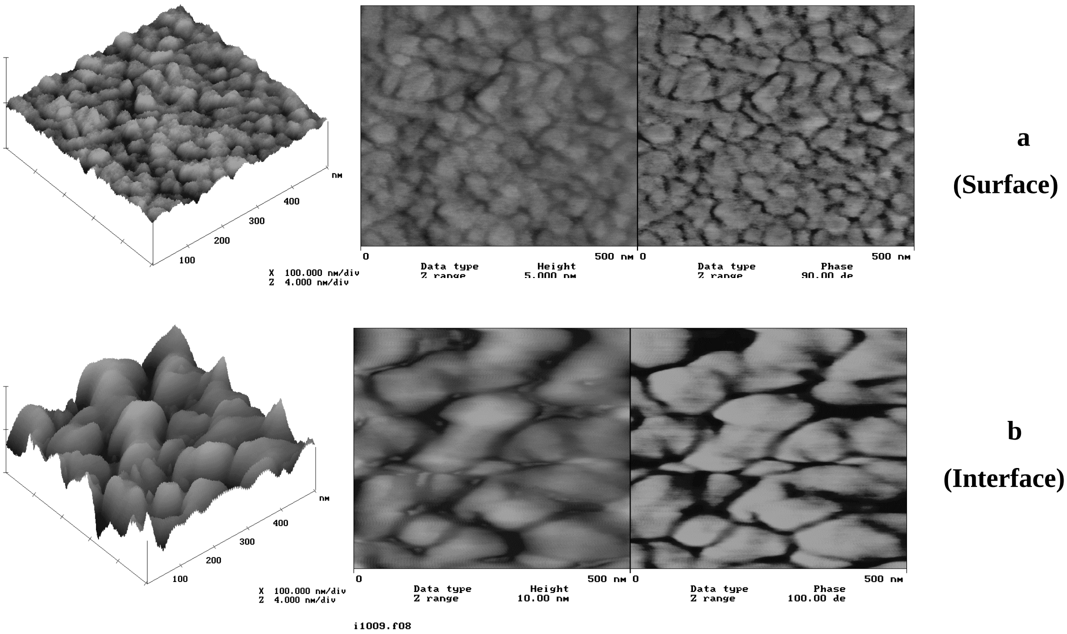



**Figure 2. 500 nm x 500 nm AFM images of the surface (a) and interface (b) of a cured amine-epoxy film applied to a silicon substrate; left: 3-D topographic; center: 2-D topographic; and right: phase. Contrast variation from black to white: a) 5 nm and b) 10 nm (topographic images), a) 90° and b) 100° (phase images).**

Another feature of Fig. 2 is the greater contrast and more well-defined microstructure of the interface as compared to that of the surface. Such differences between the surface and interface have been observed for other epoxies and coatings. The interface image clearly shows that the amine-cured epoxy has a two-phase microstructure, consisting of a matrix that appears bright and interstitial regions dispersed throughout the matrix that appear dark. The 500 nm regions that were imaged were very flat, with topographic changes of less than 10 nm for both the surface and interface. On the other hand, the phase angle difference between the bright and dark regions in the phase image of Fig. 2 is substantial ($90^{o}$ to $100^{o}$). These results indicate that the property differences between the matrix and the interstitial regions of the film are substantial.

Although the exact contrast mechanism in phase imaging is not fully understood, the bright domains in the phase image have been interpreted as due to a mechanically harder area and the dark surrounding region as due to more compliant material (Magonov et al. 1997 and Bar et al. 1997). From these assignments, the amine-cured coating shown in Fig. 2 is a heterogeneous material consisting of softer regions (dark) dispersed in a harder matrix (bright). The nodular domain in the matrix has been attributed to the high-crosslinked material and the interstitial regions as due to the less crosslinked, low molecular mass material (Cuthrell 1968, VanLandingham et al. 1999. This type of heterogeneous microstructure has been observed by AFM for other amine-cured epoxies (VanLandingham et al. 1999, Giraud et al. 2001) and polyester (Gu et al. 2001). Such inhomogeneous structure is generated during the film formation because some unreacted and partially-polymerized materials are unable to merge into the homogeneous structure and remain at the periphery of the network units [Bascom 1970]. The low crosslinked, low molecular mass region has been observed to directly correspond to the corroded spots on steel substrates (Mills and Mayne 1980). It is clear from Figure 2 that the average size of the highly-crosslinked region in the interface is larger than that on the air surface.

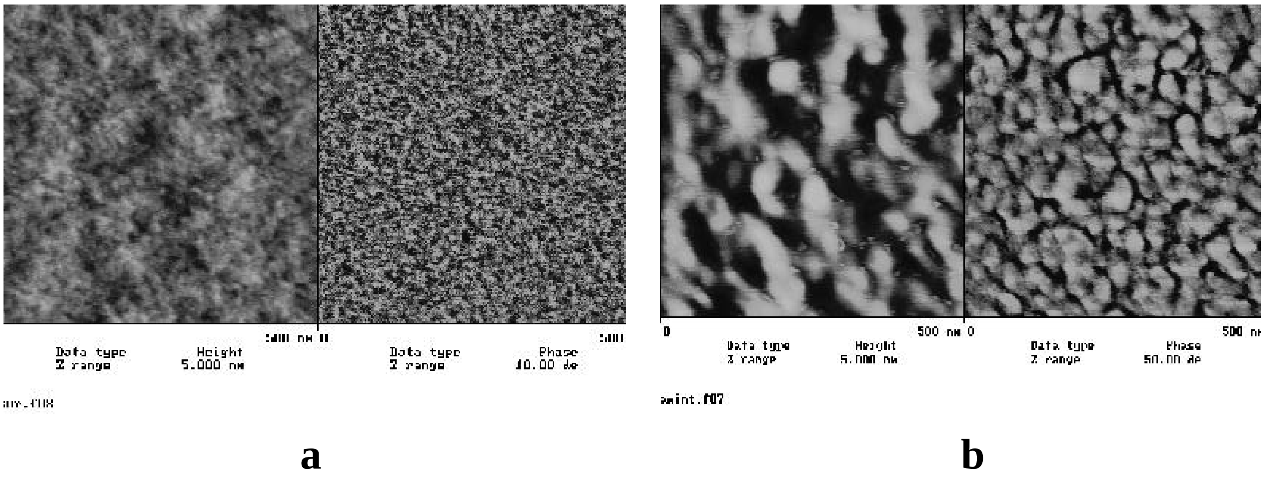
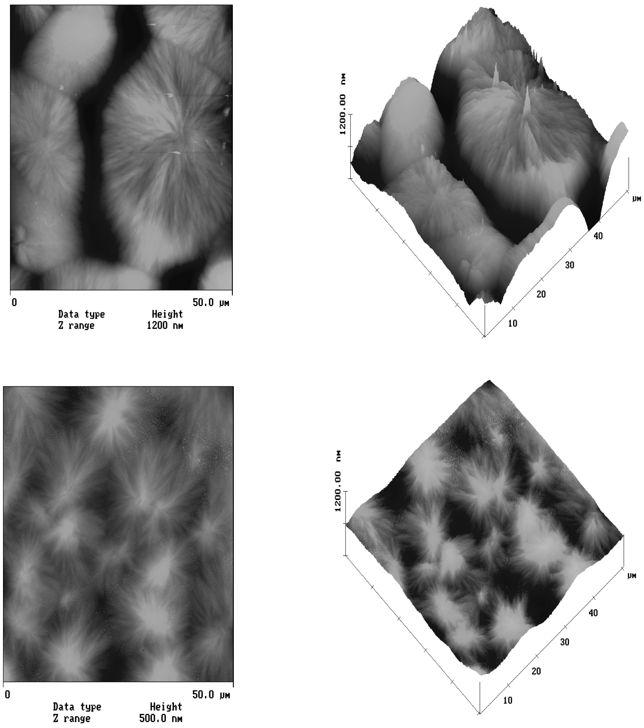



**Figure 3. AFM images of the surface (a) and interface (b) of an acrylic melamine coating applied to a silicon substrate. For both a and b, topographic image is on the left and phase image is on the right.**

AFM imaging of the air surface and film/substrate interface for acrylic-melamine coatings is shown in Figure 3. This material was a stoichiometric mixture of a hydroxy-terminated acrylic polymer and a partially-methylated melamine resin (Martin et al. 2001). Samples were prepared by drawing down the mixture on a cleaned, smooth silicon wafer. The cast film of approximately 50 μm thick was fully cured after heating at 130 $^{o}$C for 20 minutes in an air-circulated oven. Free films were obtained by peeling from the substrate after 15-minute immersion in boiled water. The films were conditioned in a $P_2O_5$-containing desiccator for one week before use. For both Figs. 3a and 3b, topographic images are on the left and phase images are on the right. As for the epoxy, the phase images show a much higher contrast and more well-defined microstructure in the interface sample than that of the surface. This difference is also indicated by a difference in the phase shift, which is only 10$^{o}$ for the air side but 50$^{o}$ for the substrate side. Further, even the phase image shows little evidence of the microstructure at the air surface. This observation and evidence of a well-defined microstructure at the interface, which is true for a variety of amine-cured epoxies and acrylic melamine coatings, suggests that the air surface of coatings could be covered with a thin layer of a lower surface-free-energy material. A thin layer of such a material should make the surface appear as homogeneous and with poor phase contrast, as observed in Figs. 2 and 3. This interpretation is consistent with extensive literature indicating that the air surface of a multicomponent polymer system is generally enriched with a lower surface-free-energy component to minimize polymer-air interfacial energy (Yoon et al. 1994 and Chen et al. 1998).

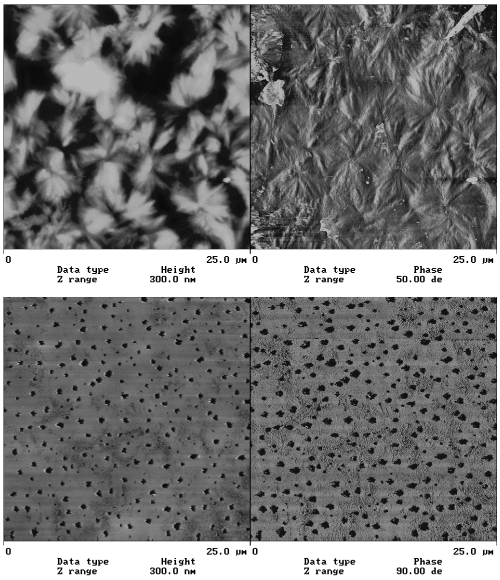
Since an enrichment of a hydrophobic layer at the surface has strong implications in many coating applications such as paintability, stain resistance, and weathering resistance, experiments to substantiate this postulation were conducted. Blends of low surface-free energy, crystallized poly (vinylidene fluoride) (PVDF) and amorphous poly (methylmethacrylate)-poly (ethylacrylate) copolymer (PMMA-co-PEA) are used for this study. These types of PVDF/acrylic polymer blends are commonly used for industrial coatings. Since PVDF is a crystallized polymer, it should be easily recognized when blending with amorphous materials. Fig. 4 displays 2-D and 3-D AFM topographic images of the surface and interface of the 70/30 (by mass) PVDF/PMMA-co-PEA) blend. These samples were prepared by applying the blend at 47 % (by mass) in isophorone solvent to glass plates. The coated plates, which had a film thickness of approximately 75 μm, were cured at 246 $^{o}$C for 10 min and cooled down slowly to 24 $^{o}$C. After a 10-min immersion in boiled water, the films were peeled off from the glass substrate. As seen in the 3-D images, the surface of these films is rougher than that of the interface and is covered almost




**a**
**(Surface)**

**b**
**(Interface)**

**Figure 4: 50 μm x 50 μm AFM topographic images of the surface (a) and interface (b) of PVDF/PMMA-co-PEA 70/30 blend. In both a and b, 2-D image is on the left and 3-D image is on the right.**




**a**
**(Surface)**

**b**
**(Interface)**

**Figure 5: 25 μm x 25 μm AFM topographic and phase images of the surface (a) and interface (b) of PVDF/PMMA-co-PEA 50/50 blend. In both a and b, topographic image is on the left and phase image is on the right. Contrast variations are indicated below the images.**

completely with PVDF crystallites. On the other hand, the interface is relative smooth and still contains some amorphous material. Further, the sizes of the crystal structure (spherulites) on the surface are substantially larger than those at the interface. These results indicate that the air surface of the blend films is enriched with low surface-free energy PVDF.

This is more evident in the AFM images of the 50/50 blend (by mass) shown in Fig. 5. The surface of this blend is largely covered with the crystallites of PVDF but the interface is void of crystal structure. Instead, the interface consists of smooth areas in between 200 nm to 300 nm deep holes. High magnification images of the interface revealed that the holes have an irregular shape and that these holes were actually the broken areas in the film (cohesive failure) resulting from the peeling. This observation indicates that the adhesion between the 50/50 blend and the glass substrate was good. Further, there is no evidence of the crystal structure of PVDF material at the interface even at 50 % mass fraction. Evidence provided from the good adhesion as well as the absence of PVDF material on the film surface adjacent to the substrate suggest that amorphous PMMA-co-PEA is preferentially present at the polymer/substrate interface.

To verify that low surface-free energy PVDF preferentially migrates to the surface and higher surface-free energy acrylic copolymer tends to move to the interface, contact angle and attenuated total reflection-Fourier transform infrared spectroscopy (ATR-FTIR) analyses were performed. Contact angle is sensitive to the first 1 nm surface layer, and for the germanium prism used in this study, the ATR technique can provide chemical information at a depth from about 0.25 μm to 1.0 μm from the surface for wavelengths of infrared radiation from 3 μm to 10 μm. Therefore, these analyses should provide information on the chemical composition at or near the surface. The results are summarized in Table 1.

**Table 1. Surface and interface wettability and ATR-FTIR properties of different PVDF/PMMA-co-PEA blends**

| | PVDF/PMMA-co-PEA blends, mass fraction | | | | | |
|---|---|---|---|---|---|---|
| | 70/30 | | 50/50 | | 30/70 | |
| | Surface | Interface | Surface | Interface | Surface | Interface |
| Water Contact Angle, degrees[1] | 71.5 ± 0.9[2] | 58.3 ± 0.5 | 72.5 ± 3.7 | 68.1 ± 4.7 | 75.1 ± 1.4 | 63.5 ± 1.9 |
| Surface Polarity | 0.21 | 0.5 | 0.17 | 0.46 | 0.16 | 0.29 |
| C-F/C=O[3] | 0.382 | 0.331 | 1.02 | 0.92 | 2.25 | 2.06 |

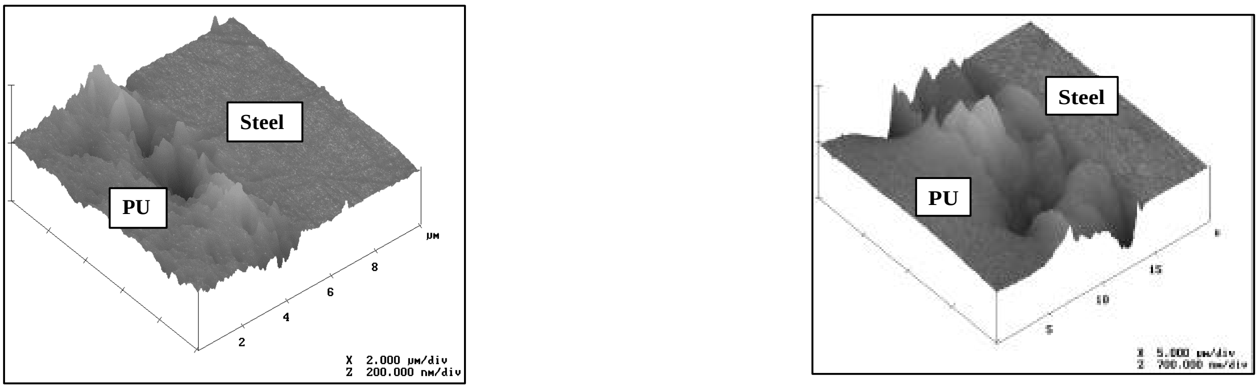
*[1]Average of six measurements; [2]The number after the ± symbol indicates one standard uncertainty; [3]FTIR-ATR intensity ratio between the C-F band and C=O band.*

In Table 1, the contact angle was measured using the sessile drop method, and the results were the average of six drops. The surface polarity is the ratio between the polar force component and the total surface free energy. These surface parameters were determined using the geometric mean method (Wu 1982) and with water and methylene iodide as the liquids. Both the contact angle and surface-free energy components, namely polar force and nonpolar force, can give a good indication of the wetting and related properties of a surface. The C-F/C=O is the ratio between the ATR-FTIR intensity of the C-F band of PVDF material and C=O band of the acrylic copolymer. All results of Table 1 show that the surface contains a larger amount of the less wettable, low surface-free energy PVDF than the interface. This observation supports the AFM results given above.

The results presented in Figs. 2 to 5, and Table 1 clearly show that a substantial difference exists in the microstructure between the surface and interface and that a low surface-free energy layer is present on the film surface exposed to air. Since the microstructure and chemical composition of a surface control its interactions with other materials and the environments, such a low surface-free energy surface should have a large effect on the wettability and weatherability of coatings. For example, the paintability and bondability to such a surface is more difficult than to more hydrophilic surfaces. On the other hand, it is a harder surface for dirts and contaminants to stick to and an easier surface for slipping (low friction). A low surface-free energy covered surface would also alter the weather resistance of the coatings. This is because, while the bulk coating underneath degrades, the more inert, hydrophobic thin layer on the surface may not undergo any change at all until much later times. This was the case for the degradation of acrylic-melamine coatings exposed to humid environments (Nguyen et al. 2001). In this study, FTIR analyses indicated substantial hydrolysis and material loss in the film within a few 200 h of exposure. However, AFM results showed little surface morphology and roughness change until after 600 h exposure. Similarly, the delay in the hydrolysis of poly(sebacic anhydride) (PSA) and poly(DL-lactic acid) (PLA) blends used to control drug deliveries has been verified by AFM and X-ray photoelectron spectroscopy (XPS) as due to a surface enrichment of the more hydrolytically-resistant PLA (Chen et al. 1998). The difference in the degradation characteristics between the surface and the bulk could explain the lack of a correlation between surface gloss loss and bulk chemical degradation, often reported in the coatings literature.

### 3.2 AFM images of coating system cross sections

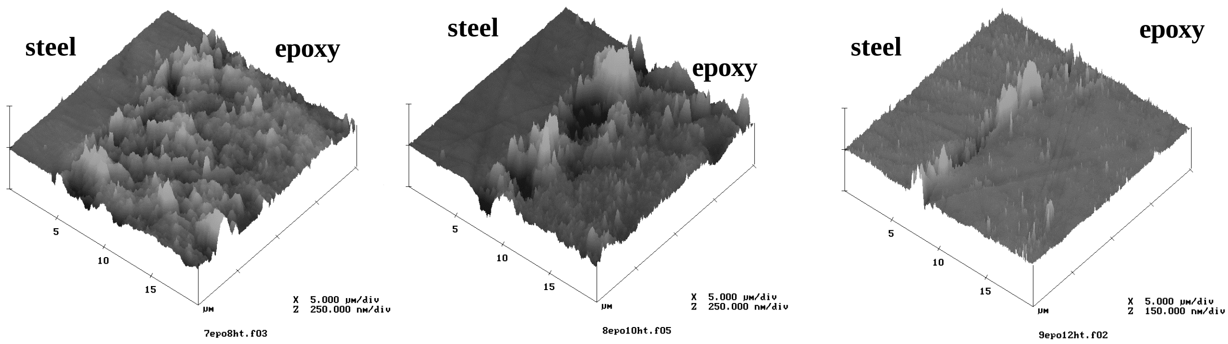
Another approach to investigate the difference between surface and the interface is to compare AFM images of the air surface with those of the cross sections. Examples for a two-part polyurethane (PU) and an amine-cured epoxy coating are illustrated in Figs. 6 and 7, respectively. These are 3-D AFM topographic images of cross sections of thick films applied to a polished steel surface. Figure 6a shows a cross-sectioned image of a PU sample cured at ambient (24 °C and 45 %RH) for two weeks, and Figure 6b is from the same PU sample but cured at 100 °C in an air-circulated oven for 2 h. The interface region (interphase) is defined as the rough section between the steel and the smooth PU matrix. The large difference between the peaks and valleys in the interphase was apparently induced by the mechanically weak material of this region and was roughened by the polishing action. These results suggest that the roughened section of these samples was not fully cured. This is supported by the smoothness of the PU matrix further away from the substrate. Such a smooth appearance indicates a more fully-cured and hard surface. It is not certain whether the weak interphase was due to an inadequate curing schedule (short time or low temperature) or due to a consumption of isocyanate by the hydrated steel surface. A post-curing study at an elevated temperature for different times could provide an answer to this question. Nevertheless, a sample containing such a weak interface region will have poor adhesion. Figure 6 also shows that the weak interface region of the heat-cured sample was narrower than that of the ambient-cured sample. These results suggest that AFM can be an useful technique to study the effect of curing schedule on the adhesion and service life of coating systems and adhesives.




a b

**Figure 6. AFM 3-D topographic images of room-temperature cure (a) and heat-cure (b) polyurethane coating on steel.**

Figure 7 shows the effect of epoxy/amine ratio on the interface region of an amine-cured epoxy coating. The epoxy resin was a 190 g/m equivalent mass material and the amine was a polyalkoxyamine. All samples were cured at 120 $^oC$ for 2 h. In the excess-amine sample (Figure 7c), the polished epoxy surface is smoother than that of the 1/1 and 1/0.8 epoxide/amine samples. Further, the rough, undercured interface region of the excess-amine sample (≈ 1.8 μm) is narrower than that of the 1/1 amine/epoxide sample (≈ 4 μm). On the other hand, the amine-deficient sample (Figure 7a) shows no distinctive interface region. The rough nature of this cross section, which is similar to the interface regions of the other two stoichiometric samples, suggests that the amine-deficient sample was not fully cured. A difference in the interphase width between the 1/1 and 1/1.2 epoxide/amine samples suggest that the interphase under-cure region was wider for the 1/1 epoxy/amine sample than that of the excess-amine epoxy. This postulation is consistent with previous reports indicating that amines from amine-cured epoxies have a tendency to migrate and preferentially adsorb on glass fiber (Palmese and McCullough 1992) and hydrated steel (Wicks 1987) surfaces. It is noted that, under ambient condition, a layer of iron oxides approximately 35 nm to 40 nm is instantaneously formed on the steel surface. Since iron oxides are highly energetic, they adsorb water rapidly. Thus, it is expected that the surface of the polished steel used in this study was covered with a hydrated oxide layer. The migration of amine to the substrate surface not only leaves the material near the interface under-cured, which results in weak adhesion, but also produces an interface with an excess of amine. Since low molecular mass amines are hygroscopic, the presence of such material at the interface will introduce an osmotic pressure when this material is exposed to water or high relative humidities. This will result in an increase in the rate of water transport to the interface and an increase of the thickness of the water layer at that location. Thus, if the substrate is not properly treated or the coating is not properly formulated, the interface of amine-cured coated materials in humid environments is susceptible to water attack, resulting in loss of adhesion and delamination.




a b c

**Figure 7. 15 mm x 15 mm AFM 3-D topographic images of amine-cured epoxy coated steel having different epoxide/amine ratios; a) epoxide/amine: 1/0.8; b) epoxide/amine: 1/1; and c) epoxide/amine:1/1.2.**

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## 4 CONCLUSIONS

The surface and interface microstructure of a variety of coatings applied to different substrates have been characterized with nanoscale resolution by tapping mode atomic force microscopy (TMAFM). Contact angle measurements and attenuated total reflection-Fourier transform infrared spectroscopy were performed on some coating systems to verify AFM data. In addition, the effects of curing conditions and stoichoimetry on the interface region were also studied for polyurethane and epoxy coatings. The results show that the surface exposed to the substrate has a much higher contrast and a more well-defined morphology than those of the surface exposed to air. This result suggests that there is a substantial difference in both chemical composition and microstructure between the surface and interface, and that a thin layer of a low surface-free energy material generally covers coating surface. These results have strong implications for many phenomena occurring on the surface of coating systems, including the wettability, bondability, paintability, and weatherability.

***Certain commercial product or equipment is described to specify adequately the experimental procedure. In no case does such identification imply recommendation or endorsement by NIST, nor does it imply that it is necessarily the best available for the purpose*.

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# Critical Review Of Corrosion Deterioration Models For Reinforced Concrete

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**Summary**: **Corrosion of embedded reinforcing steel is considered the major mode of deterioration for concrete structures. Traditionally, much attention has been focused on models which can predict the time to activation of the corrosion process. However, in some circumstances the propagation stage of corrosion may contribute significantly to the service life. Consequently, there is an urgent need for numerical models which are capable of describing the rate of attack.**

**The paper focuses on localised attack of reinforcing steel resulting from chloride contamination of the concrete cover. Three commonly used deterioration models are reviewed and an improved model for localised corrosion is presented. This model takes into account the accelerating effect of macrocells on local corrosion rates assuming a cylindrical shape of the pitting site. The results are presented in terms of local penetration depth, surface area under corrosion attack, and steel volume consumed by corrosion. The improved corrosion deterioration model can be used to formulate a probabilistic model which allows calculation of the probability that a given reduction of the reinforcement cross-section has taken place at a given time.**

**Keywords. steel reinforcement, concrete, corrosion, durability, modelling, service life**

## 1 INTRODUCTION

Whole life performance assessment is gradually becoming a requirement for the design of new structures and for inspection, repair, upgrading, replacement, and demolition decisions of ageing and deteriorated structure. This has also urged for design for durability of steel reinforced concrete structures. Several deterioration models are currently being used in service life prediction and consequently can be applied for engineering design and maintenance purposes.

Corrosion of reinforcing steel in concrete is generally regarded as the most prevalent mode of premature distress and deterioration of structural concrete. This has resulted in a clear need from both the industry and field of research that a model be developed to determine the initiation of corrosion and to predict the remaining service life of corroded reinforced concrete structures. The conceptual model proposed by Tuutti (1982) is frequently used as a basis for the assessment of the loss of serviceability of reinforced concrete during its service life (DuraCrete 1998). This model comprises two stages, i.e. the initiation stage which involves the penetration of deleterious substances through the concrete cover, and the propagation stage which involves active corrosion following steel depassivation (See Fig. 1). The expected service life will include the time needed to corrode the steel to an extent that renders the structure unsuitable for further service.

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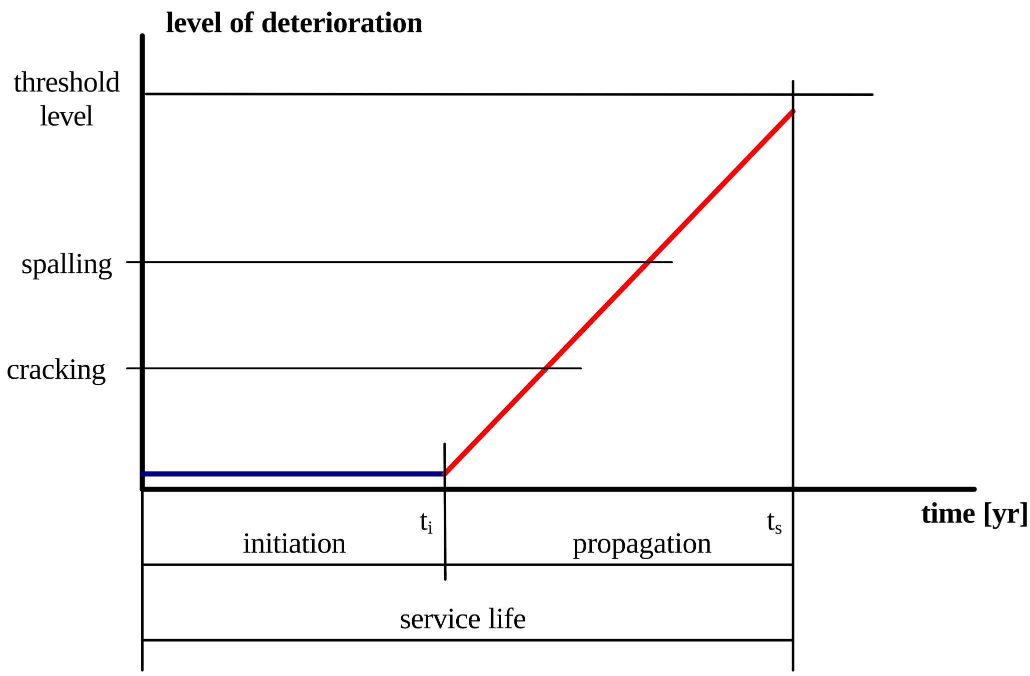


**Figure 1. Conceptual deterioration model showing the stages during the service life of a reinforced concrete structure with respect to reinforcement corrosion.**

Most of the durability models currently available are focusing on the prediction of corrosion initiation. However, it has to be borne in mind that initiation of corrosion represents no immediate threat to the structural integrity of the structure. Consequently, service life predictions based on initiation of reinforcement corrosion result into a conservative approach since the propagation stage may contribute significantly to the service life (Gulikers 2000).

In recent years several deterioration models describing corrosion attack of reinforcing steel have become available(e.g. Andrade *et al.* 1989, Roberts *et al.* 2000). However, most of these models show considerable drawbacks and pitfalls which necessitates a critical review. Moreover, these models conflict with the behaviour generally encountered in real structures. This paper focuses on the propagation period of reinforcement corrosion and addresses issues associated with localised attack resulting from contamination of the concrete cover by penetrating chlorides.

## 2 STRUCTURAL CONSEQUENCES OF REINFORCEMENT CORROSION

Damage resulting from reinforcement corrosion is characterised by:

- surface staining;
- cracking or spalling of the concrete cover;
- delamination of the concrete cover;
- reduction of the cross sectional area of the reinforcing steel;
- reduction of the steel-concrete bond.

Furthermore there exists some experimental evidence that corrosion results into a decrease in elongation at maximum load and impair the mechanical properties of the steel ((Andrade *et al.* 1991). In general, the impact of corrosion-damaged reinforcement on performance of reinforced concrete structures is associated with reductions in member stiffness, ductility, ultimate strength and fatigue behaviour. Consequently, corrosion of reinforcing steel is associated with the serviceability as well as the ultimate limit state of a concrete structure or element. The first corresponds to a limiting condition resulting from damage such as excessive corrosion-induced cracking or deflections . The second limit state is related to a loss of load-bearing capacity due to the loss of steel reinforcement cross section, or bond between the concrete and reinforcing steel.

Generally, the basic problem associated with the deterioration of reinforced concrete due to corrosion

is not that the reinforcing steel itself is reduced in mechanical strength, but rather that the products of corrosion generate stresses within the concrete which cannot be supported by the limited plastic deformation of the concrete, and the concrete therefore cracks. This results into a weakening of the bond and anchorage between concrete and reinforcement which directly affects the serviceability and strengths of concrete elements within a structure (Cabrera 1996).

Steel corrosion products (rust) usually occupy a much larger volume than the uncorroded steel and consequently generate stresses that eventually cause cracking and spalling of the concrete cover. This makes it easier to for aggressive agents to ingress towards the steel, with a consequent increase in the rate of corrosion. The strongly localised progress of corrosion reduced the cross-sectional area of the steel, thus impairing its load- carrying capacity. However, it is argued that the overall effects of localised pitting corrosion on the load bearing capacity of a bridge deck may be no more severe than the effects of more widespread general corrosion. Furthermore, for large structures such as bridge decks, even propagation of corrosion at a single point only will have a limited effect on its load-bearing capacity.

The amount of corrosion required to initiate cracking is dependent on the quality of the concrete cover and the ratio of cover to reinforcing bar diameter. According to Broomfield (1997) visible cracks due to corrosion appear in the concrete cover after reinforcing bar section losses of 10-100 µm. Formation of cracks, especially those that form along the steel reinforcement, probably provides the best visible indicator for use in corrosion assessments and trying to develop a relationship between corrosion and performance (Naus *et al.* 2000). The deposition of corrosion products at the steel-concrete interface together with cracking of the concrete cover also result in a reduction of bond between reinforcing steel and concrete, thereby impairing the bar anchorage.

The characteristics of corrosion resulting from carbonation and chloride ingress differ in one principal aspect. Carbonation-induced corrosion tends to be general whereas chloride-induced corrosion is characterised by local, rapidly corroding areas of bars.

In the following chapters some models on corrosion attack will be reviewed and an improved model for localised corrosion will be presented.

## 3 EXISTING MODELS OF CORROSION ATTACK

### 3.1 General

The corrosion rate is most frequently given /supplied as an averaged corrosion current density, $i_{corr}$, expressed in mA/m$^2$ but for practical reasons a conversion into µm/year is more appropriate. This conversion can be accomplished by using Faraday's law:

$$1\ \mathrm{mA/m^2} \equiv 1.16\ \mu\mathrm{m/yr} \qquad \text{or} \qquad 1\ \mu\mathrm{A} \equiv 1\ \mathrm{mm^3/yr}$$

From practical point of view a corrosion rate of less than 1 to 2 mA/m$^2$ might be considered negligible since it is normally related to the current to maintain a passive oxide film.

Once corrosion has started, i.e. after depassivation by carbonation or chloride-contamination of the concrete cover, the rate is dependent on the continuous supply of water and oxygen and the temperature at the steel section. Average corrosion current densities of 10-30 mA/m$^2$ are frequently encountered in practical situations with active corrosion. Only seldomly values of 100 mA/m$^2$ or higher are determined. In some rare instances of low oxygen availability corrosion can occur, but will not result in expansive corrosion products. This is known as "black rust" which represents a serious condition since there is no visual warning.

In the following some common corrosion deterioration models will be reviewed.

### 3.2 Uniform corrosion model

Generally, values for $i_{corr}$ of reinforcing steel are determined indirectly by experiments on site, e.g. by means of polarisation resistance or galvanostatic pulse techniques. Usually, the corrosion rate values thus obtained are expressed as surface-averaged corrosion rates although significant spatial variations may be present. Consequently, uniform corrosion is most commonly assumed when calculating corrosion penetration rates of reinforcing steel bars (Vu and Stewart 2000). According to this approach uniform corrosion attack occurs on the complete surface of a steel bar. Consequently, the geometrical shape of the rebar cross sectional area is considered to remain unchanged as a circle of gradually decreasing diameter during the propagation stage of corrosion.

Assuming uniform corrosion attack, the residual diameter of the reinforcement bar, $\varnothing_{res}$, can be calculated:

$$\varnothing_{res} = \varnothing_{rebar} - \lambda \cdot i_{corr} \cdot t_p \qquad \text{[mm]}$$

$i_{corr}$ average annual corrosion current density [mA/m$^2$];

$\lambda$ factor to convert average corrosion densities to average penetration rates:

$\lambda = 2.3294 \cdot 10^{-3}$ [mm/(mA/m$^2$)];

$t_p$ corrosion propagation time; $t_p = t_s - t_i$ [yr] is the time elapsed since corrosion initiation;

$t_s$: service age of the structure [yr];

$t_i$ length of initiation period [yr].

However, it is unlikely that the complete circumference of a steel bar is under corrosion attack once the depassivation front reaches the level of the reinforcement. Bearing in mind that the depassivation front gradually moves into the interior of the concrete body, the depassivated steel surface increases in time, at least theoretically. However, the penetration rates for a partly depassivated rebar circumference may be considerably higher than the average corrosion rate values usually given in literature due to macrocell effects. These influences are elaborated by Gulikers (2002) which resulted into an improved propagation model for carbonation-induced corrosion taking into account a time-dependent moving depassivation front.

### 3.3 Pitting corrosion model

In order to take account of the localised nature of chloride-induced corrosion, the uniform corrosion model has been modified by introducing a pitting factor, α, and defining a critical limiting pit depth (Andrade *et al.* 1996). Accordingly, the critical residual bar diameter, $\varnothing_{res}$, can be obtained from the initial bar diameter, $\varnothing_{rebar}$, and the local reduction of the cross section

$$\varnothing_{res} = \varnothing_{rebar} - \alpha \cdot \lambda \cdot i_{corr} \cdot t_p \qquad \text{[mm]}$$

According to González *et al*. (1995) the maximum pit depth due to localised attack will be approximately 4 to 10 times the surface-averaged uniform attack. This simplified approach has been adopted in the probabilistic method of design for durability developed within the DuraCrete Project (1998).

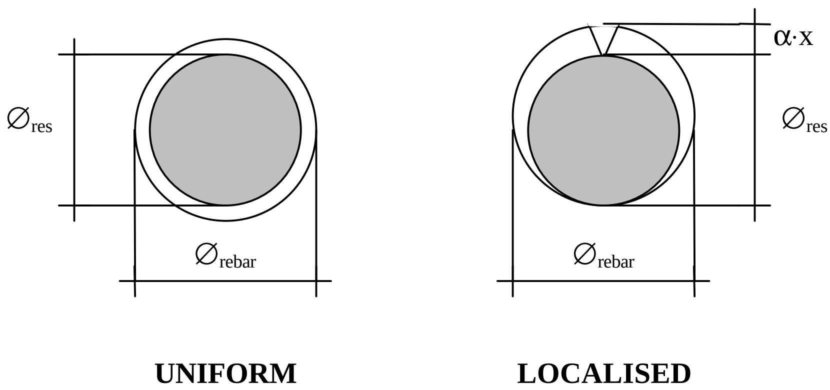


**Figure 2. Residual reinforcing bar diameter due to uniform and localised corrosion attack.**

Although this approach is practical it lacks a sound scientific basis. Moreover, the magnitude of the pitting factor has to be chosen on arbitrary grounds.

### 3.4 Wavefront model

The influence of a time-dependent depassivation front is taken into account by a “wavefront” type progression of corrosion through the reinforcing bar (Middleton and Hogg, 1998). This model assumes that the corrosion front is in a direct line with the front of corrosion attack. Both front lines run in parallel with the exposed concrete surface. The corrosion penetration rate and consequently the steel volume consumed by corrosion are therefore dependent on the chloride penetration rate. Initiation of corrosion starts at the steel surface nearest to the exposed concrete surface and will gradually extend to surface areas located at greater depths. The wavefront model has been adopted by Pedersen and Thoft-Christensen (1993) for a reliability analysis of corroded tendons in prestressed concrete.

The residual cross sectional area, $A_{res}$, is mathematically expressed by:

$$A_{res} = (\pi + \sin\theta \cdot \cos\theta - \theta) \cdot r_{rebar}^2 \qquad \text{[mm}^2\text{]}$$

with

$$\theta(t) = \arccos\left(\frac{r_{rebar} + c - x_{dep}(t)}{r_{rebar}}\right) \qquad [-]$$

$r_{rebar}$ initial radius of the rebar [mm];

$c$ (local) thickness of the concrete cover [mm];

$x_{dep}(t)$ depth of the depassivation front [mm].

This approach has been exemplified for a concrete cover quality characterised by a range of effective chloride transport coefficients. It is assumed that the chloride penetration into the concrete cover is described by:

$$c_{crit} = c_{surf} \cdot \left[1 - \mathrm{erf}\left(\frac{x_{dep}}{2\sqrt{D_{Cl} \cdot t}}\right)\right] \qquad [\%]$$

with $c_{crit}$ critical chloride concentration at which steel depassivation occurs [%];

$c_{surf}$ fixed chloride concentration a the external concrete surface [%];

$x_{dep}$ depth of the depassivation front [m];

$D_{Cl}$ effective chloride transport coefficient [$m^2/s$];

$t$ exposure time [s].

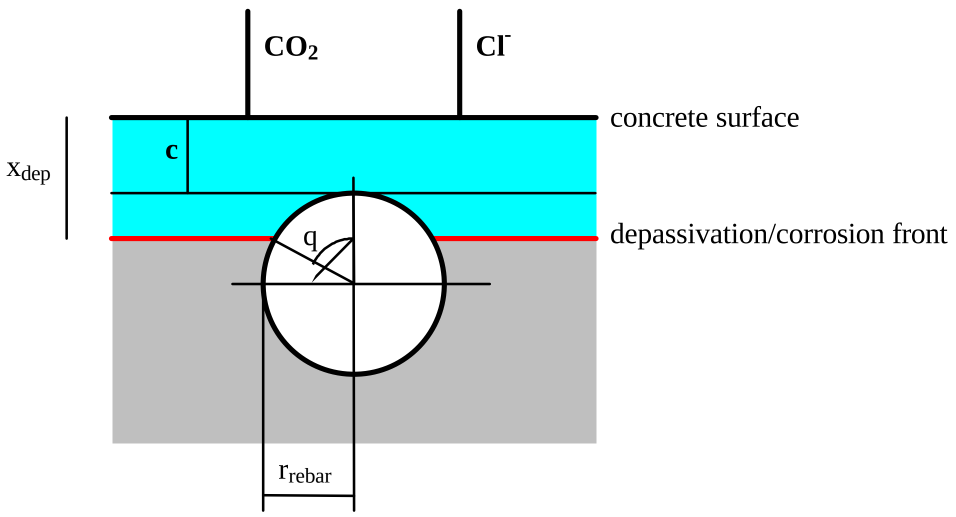


**Figure 3. Wavefront model for corrosion penetration of a reinforcing steel bar.**

Through modification an expression for the time-dependent depth, $x_{dep}$, of the depassivation front can be obtained:

$$x_{dep} = 2D_{Cl} \cdot A \cdot \sqrt{t} \qquad [m]$$

with $\mathrm{erf}(A) = \left(1 - \frac{c_{crit}}{c_{surf}}\right)$, considered to be constant.

According to the wavefront model the penetration rate of the depassivation front, $v_{dep}$, equals the steel corrosion penetration rate, $v_{corr}$, which is given by:

$$v_{dep} = v_{corr} = 31.56 \cdot 10^9 \cdot \frac{D_{Cl} \cdot A}{\sqrt{t}} \qquad (mm/yr)$$

Following the wavefront model, if the average cover depth is known and the time to corrosion initiation is derived from frequent visual observations, the corresponding corrosion penetration rate could be estimated from:

$$v_{cor} = 31.56 \cdot 10^9 \cdot \frac{c}{2 \cdot t_i} \qquad (mm/yr)$$

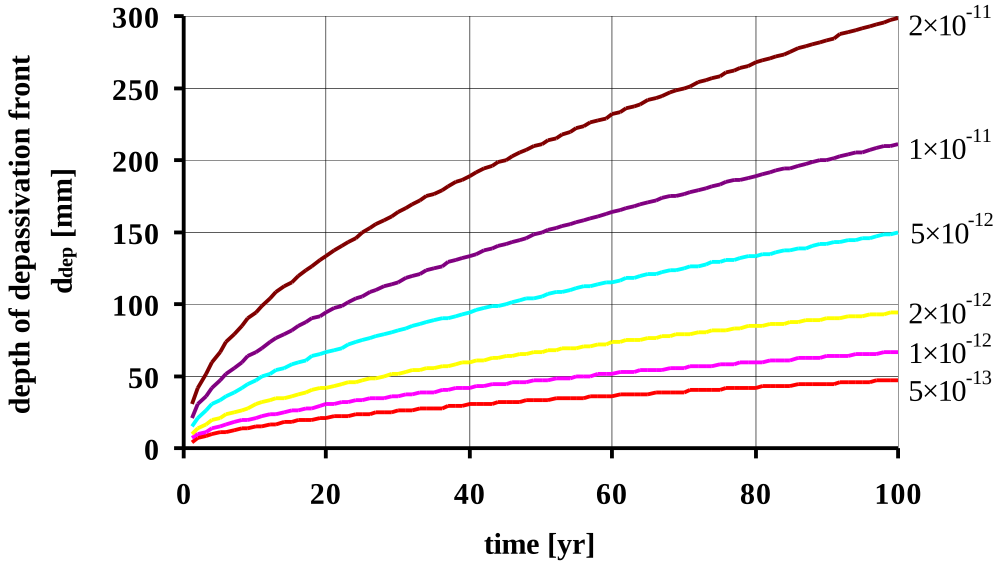


**Figure 4. Wavefront model: depth of depassivation front as a function of time for different values of $D_{Cl}$ ($m^2/s$).**

The wavefront model seems to predict unrealistically high penetration rates: for a cover depth of 20 mm the penetration rate ranges from 0.560 to 10 mm/yr, for effective chloride transport coefficients, $D_{Cl}$, of $5{\cdot}10^{-13}$ to $2{\cdot}10^{-11}$ $m^2/s$, respectively.

Moreover, the implicit assumption that corrosion will attack on the complete length of a reinforcing bar is not plausible for chloride-induced corrosion. On the other hand, the calculated values may be considered as local penetration rates.

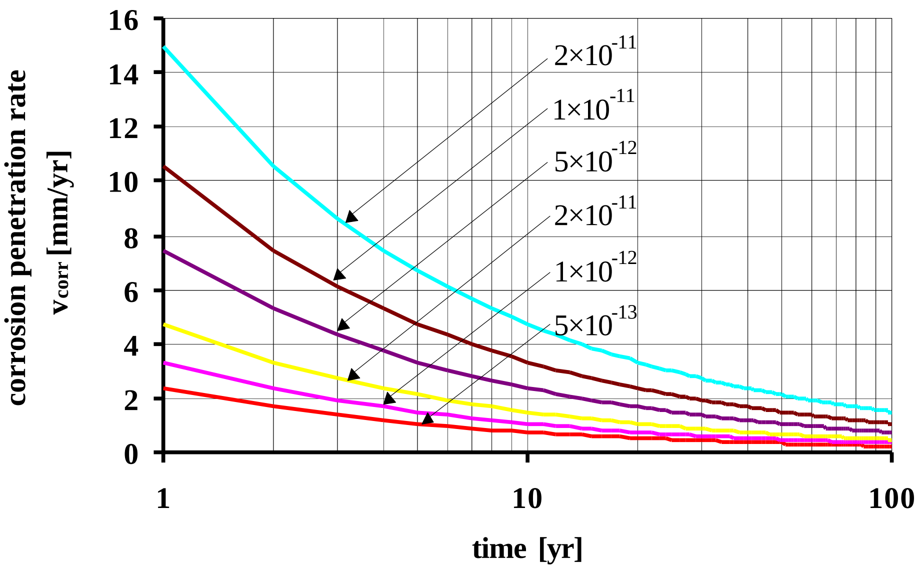


**Figure 5. Wavefront model: Corrosion penetration rate as a function of time for different values of $D_{Cl}$ ($m^2/s$).**

## 4 IMPROVED MODEL FOR LOCALISED CORROSION ATTACK BY CHLORIDES

The pitfalls of the models described above served as the main motive to develop improved deterioration models for both carbonation and chloride-induced reinforcement corrosion (Gulikers 2002). However, with respect to structural safety of reinforced concrete structures only corrosion induced by chloride-contamination is considered of practical relevance.

Chloride ions present within the concrete matrix tend to depassivate embedded steel by locally breaking down the protective layer of iron oxides. This results into the development of macrocells which are characterised by distinct and spatially fixed cathodic and anodic areas. This situation gives rise to a unfavourable cathode to anode area ratio inducing high local corrosion rates. The intense anodic areas so produced develop into characteristic pits which may continue to grow provided the chloride ion concentration exceeds a critical level.

The deterioration model developed for chloride-induced corrosion assumes that the local corrosion rate is determined by the cathodic polarisation behaviour of the passive steel surrounding the pitting area. The macrocell current produced is dependent on the throwing power, the availability of oxygen and moisture, the thickness and quality of the concrete cover, and the geometry and size of the reinforcing steel. In (Gulikers 2001) a numerical model has been developed for so-called coplanar and planparallel macrocells. This model allows calculation of the macrocell current flowing between anodic and cathodic steel surfaces, taking into account all parameters of practical relevance.

Although the macrocell current will reflect seasonal variations in the moisture content of the concrete cover, it may be assumed that it's averaged annual magnitude is almost constant over time for given geometrical and exposure conditions.. Consequently, this average value of the macrocell current can be used as an input to calculate the loss of cross-sectional area of the steel bar during the propagation period.

For simplicity, the pits resulting from localised corrosion by chloride contamination are assumed to have a cylindrical shape. The pitting site starts as tiny spot a the top surface and gradually expands in radial direction into the body of the steel rebar. The steel volume consumed by corrosion and the steel surface under corrosion attack is obtained from a numerical analysis of the intersection of two cylinders, one being the steel rebar (see Fig. 6).

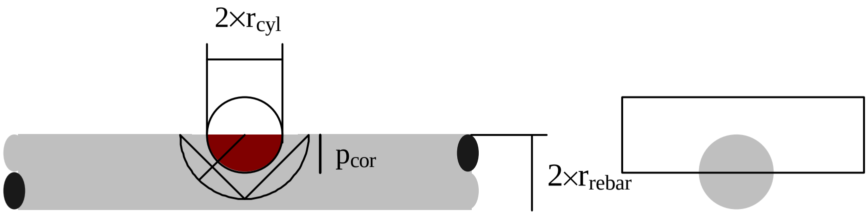


**Figure 6. Improved model for localised corrosion attack**

A curve-fitting exercise was performed on the results of the numerical analysis. This resulted in a best-fit polynomial mathematical expression describing the steel volume consumed by corrosion, $V_{corr}$, and the penetration depth, $p_{corr}$, respectively, with the ratio $r_{cyl}/r_{rebar}$ as the main variable:

$$R = A + B\cdot X + C\cdot X^2 + D\cdot X^3 + E\cdot X^4 \qquad (–)$$

with

$$R = \frac{A_{corr}}{r_{rebar}^2} \text{ or } R = \frac{V_{corr}}{r_{rebar}^3}$$

$$X = \frac{r_{cyl}}{r_{rebar}}\text{; } 0 \le X \le 2.0$$

The constants A to E obtained from fitting procedures are given in Table 1.

**Table 1. Fitting constants for calculation of the corroding steel surface, $A_{corr}$, and the steel volume consumed by corrosion, $V_{corr}$, respectively**

| *Fitting constant* | $A_{corr}$ (–) | $V_{corr}$ (–) |
|---|---|---|
| A | $-1.018\cdot10^{-1}$ | $-1.356\cdot10^{-3}$ |
| B | +3.086 | $-5.1424\cdot10^{-2}$ |
| C | +3.077 | +1.18035 |
| D | $-1.868\cdot10^{-1}$ | $+1.69018\cdot10^{-1}$ |
| E | $-5.1005\cdot10^{-1}$ | $-0.4974\cdot10^{-1}$ |

A comparison between the results of the numerical analysis and the fitting curve is presented in Fig. 7.

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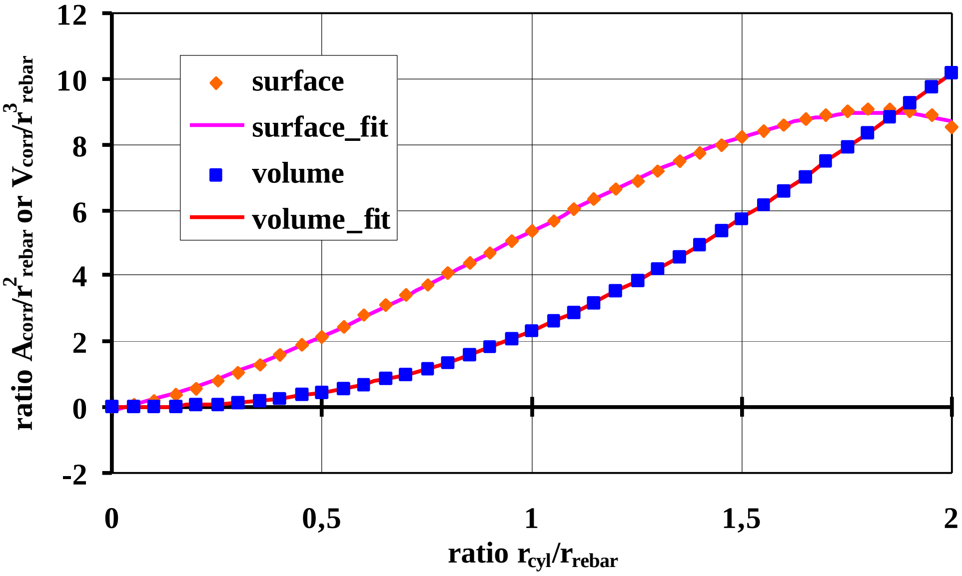


**Figure 7. Relationship between $r_{cyl}/r_{rebar}$ and $A_{corr}/r^2_{rebar}$ or $V_{corr}/r^3_{rebar}$, respectively.**

The steel volume consumed by corrosion shows a continuous increase with a maximum ratio of 10.17, whereas the steel surface area under attack demonstrates a maximum ratio of 9.07 for $r_{cyl}/r_{rebar} \approx 1.84$.

Under most exposure conditions, the magnitude of the macrocell current may vary considerably dependent on the thickness and quality of the concrete cover, and the diameter of the steel rebar. According to calculations performed by Gulikers and Raupach (1999) for coplanar arrangements macrocell currents may range from 35 to 2738 µA, corresponding to a rate of steel removal of 41 and 3206 $mm^3$/year, respectively. For a given macrocell current, the steel volume consumed, surface area under corrosion attack and the penetration depth can be calculated as a function of time.

With respect to structural behaviour the corrosion penetration depth, $p_{corr}$ is of importance. Fig. 8 shows the range of pit penetration depths for a reinforcing bar with a diameter of 16 mm attacked by macrocell currents ranging from 50 to 5000 µA.

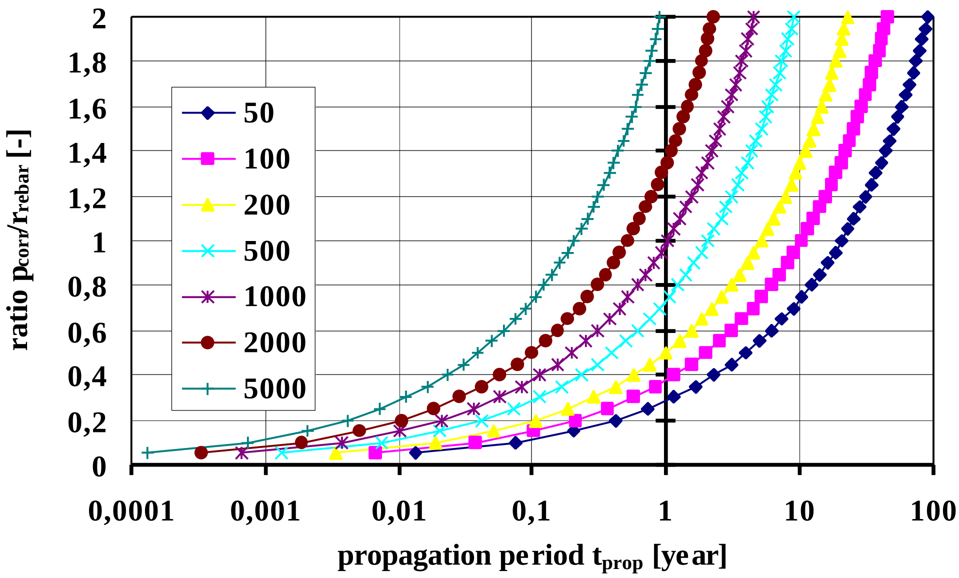


**Fig. 8. Ratio between $p_{corr}/r_{rebar}$ and propagation period for a steel bar Æ16 mm attacked by macrocells ranging from 50 to 5000 mA.**

The propagation periods thus calculated range from 0.9 and 90 years for macrocell currents ranging from 50 to 5000 µA.

The instantaneous corrosion penetration rate expressed in mm/yr can be calculated based on the steel surface area available within the pit crater. A minimum value is obtained for $r_{cyl}/r_{rebar} = 1.84$ where the surface ratio is at its maximum. The calculated results shown in Fig. 9 are presented on a double logarithmic scale. The difference between the initial values and the values at the end of the propagation period may differ by two to three orders of magnitude. Note that the initial values correspond to $p_{corr}/r_{cyl} = 0.05$ which may severely underestimate the actual initial rate of attack.

These examples clearly demonstrate that the improved model allows calculation of the local penetration depth and corrosion rate as a function of time. The values can serve as an input for models describing the structural performance of reinforced structures.

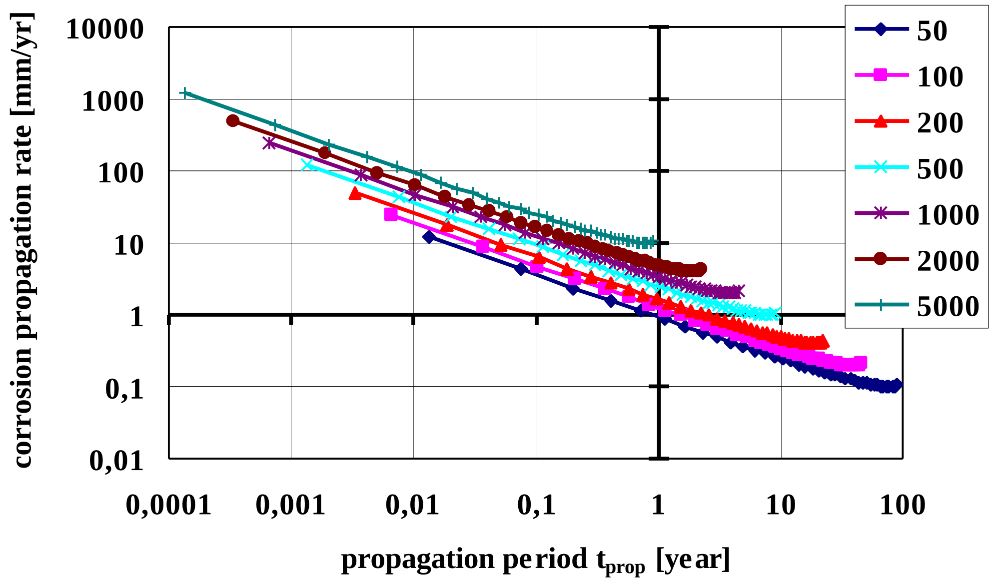


**Figure 9. Local corrosion propagation rate as a function of time**

## 5 CONCLUSIONS

The improved model seems to be suitable to simulate the effects of localised corrosion attack. The model reflects the electrochemical behaviour characterised by the development of macrocells normally encountered in concrete environments contaminated by chlorides. This approach allows calculation of the loss effective cross sectional area and corrosive expansion of the bar.

However, a critical evaluation of the assumptions being made in the development of the model is required. Especially this applies to the geometrical shape of the pit and the magnitude of the macrocell. Information on the true morphology of corrosion attack by chlorides for a range of exposure conditions would therefore aid in modifying the model to a more realistic level. Moreover, reliable quantitative information on macrocell currents is lacking since corrosion rate measurements performed on site do not allow a distinction to be made between micro- and macrocell corrosion. The improved attack model can be used to formulate a probabilistic model which allows calculation of the probability that a given reduction of the reinforcement cross-section has taken place at a given time. Therefore, the presentation of the improved corrosion deterioration model is considered to be of practical importance for further discussion.

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# New Prediction/Evaluation Methods Of Service Life Of Reinforced Concrete With Polymeric Coatings

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**Summary: Physical service life of reinforced concrete (RC) buildings in the ordinary atmospheric surroundings is reasonably determined by the time when the occurrence of crack and spalling of cover concrete makes damage to daily safety as the results of the progress of carbonation of concrete and the accompanying corrosion of reinforcing steel, except for the special cases of deterioration due to salt damage in the severe marine environment and frost damage in the cold climates. If we make effective use of surface finishing materials, however, physical service life of RC buildings can be prolonged by their suppressive effects against the progress of carbonation of concrete and corrosion of reinforcing steel.**

**In this paper, I propose the rational setting methods of physical service life of reinforced concrete with surface finishing materials based upon quantitative evaluation for their experimentally and theoretically confirmed suppressive effects, also by setting the equivalent thickness of cover concrete and the equivalent corrosion losses of reinforcing steel due to their suppressive effects.**

**On the other hand, linear law experimentally confirmed as the results of accelerated carbonation test under step response is discussed as the new prediction/evaluation methods of the progress of carbonation of concrete modelling the tendency of the increase in the concentration of atmospheric carbon dioxide as one of the global environmental problems.**

**Keywords: Reinforced Concrete, Polymeric Coating, Carbonation, Corrosion, Suppressive Effects**

## 1 INTRODUCTION

Service life of building materials used in given elements of given buildings should be as long as possible during in service from the viewpoint of the conservation of resources and the preservation of energy, considering the reduction of the burdens to the environment. However, the compatibility of each service life should be considered, because structural materials and components should satisfy as long durability as possible during the designed service life, but non-structural or finishing materials such as polymers and cement boards etc. are considered to be renewed or exchanged at certain intervals in order to maintain the good condition of buildings as a whole. Except for the special fast deterioration due to salt damage in the severe marine environment and frost damage in the severe cold climate, deterioration of reinforced concrete (RC) components and / or buildings in the ordinary atmospheric environment in the mild climate progresses through the carbonation of concrete and the accompanying corrosion of reinforcing steel. It is generally accepted that the physical service life is set by the time of crack and spalling of cover concrete by the expansion pressure of rust products ($t_2$ in Fig.1 a), because the daily safety is severely damaged. Fig.1 shows this concept adopted in Japan Architectural Standard Specification of Reinforced Concrete Work JASS5 (Architectural Institute of Japan, 1982) compared with the service life of continuous fiber reinforced concrete (FRPRC) as newly prevailing building structural components ($t_1$ in Fig.1 b). Recently in Japan, there are many news which make many people feel once again “Concrete Crisis” due to the severe deterioration of reinforced concrete structures, so that rational setting methods of physical service life of RC components and / or buildings are becoming all the more needed again. On the other hand, though the progress of carbonation has been discussed until now considering the concentration of atmospheric carbon dioxide ($CO_2$) as the direct external degradation factor as a constant value of about 300ppm, it was observed to have increased year by year with the seasonal variation in the latter half of twentieth century (Special Report of the Government of U.S.A., 1981), and if no efforts are to be done to reduce it, it will increases at a drastic speed in the twenty-first century. This tendency of the increase of the concentration of atmospheric $CO_2$ attracts the attention of the people all over the world as one of the serious global environmental problems causing global warming. However, it is anticipated that this tendency will accelerate the progress of carbonation of concrete from the viewpoint of durability of reinforced concrete structures. Therefore, it is necessary in the future to establish the rational prediction and evaluation methods of the progress of neutralization of concrete taking into account this tendency (Fukushima 1990, Fukushima 1994, Fukushima et al. 1995, Fukushima et al. 1996, Fukushima et al. 1997). When we view the real situations of reinforced concrete (RC) buildings, however, some surface finishing due to polymeric coating materials etc. have been done, and off-form style of RC buildings are very rare. Consequently, we never fail to take into consideration the influences of surface finishing materials such as polymeric coatings, when we discuss the physical service life of RC components and / or buildings based upon the progress of carbonation and the accompanying corrosion of reinforcing steel (Fukushima & Motohashi 1985), even if we have to take into account that they are liable to deteriorate under outdoor conditions due to weathering (Fukushima, 1986).This paper deals with the rational setting methods of service life of RC components and/or buildings considering the suppressive effects of polymeric surface finishing materials against the carbonation of concrete and the corrosion of reinforcing steel experimentally confirmed, and also taking into account of the increasing tendency of the concentration of atmospheric CO2 .

## 2 RESEARCH METHODS

When we do effective surface treatment by polymeric surface coatings, we can expect the elongation of physical service life of RC components and/or buildings due to their suppressive effects against carbonation of concrete and the corrosion of reinforcing steel.

Firstly, I would like to explain the evaluations methods of the suppressive effects of polymeric surface coatings against the carbonation of concrete, which were confirmed experimentally and by the results of field research, and the conversion methods of their suppressive effects into equivalent thickness of cover concrete (Fukushima & Fukushi 1997).

Secondly I would like to explain the evaluation methods of the suppressive effects of polymeric surface coatings against the corrosion of reinforcing steel which were conformed by accelerated corrosion test and outdoor and indoor exposure tests, and the conversion methods of their suppressive effects into equivalent corrosion losses of crack occurrence.

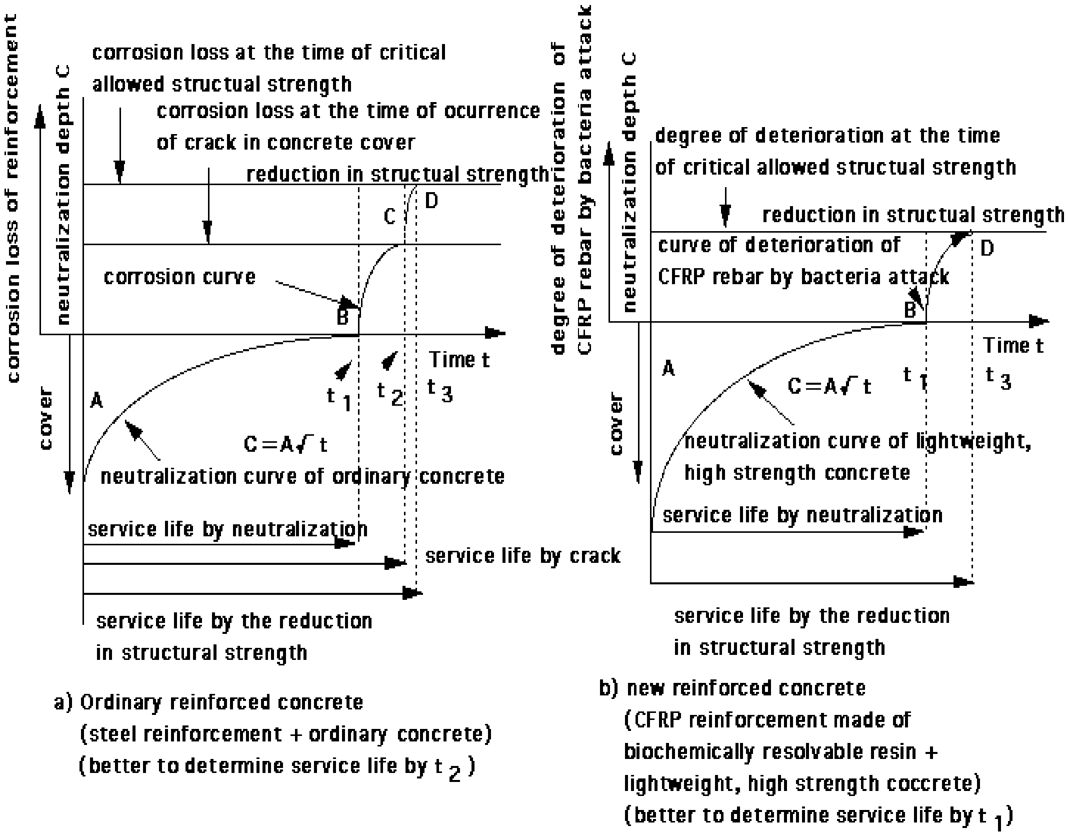


**Figure1 Comparison of Concepts of Prediction of Service Life between New Reinforced Concrete with CFRP Reinforcement and Ordinary Reinforced Concrete with Steel Reinforcement**

Further by combining the two suppressive effects of polymer surface coatings, I would like to propose the rational setting methods of service life of RC components and/or buildings with polymeric surface finishing materials (Fukushima, 1990).

On the other hand, I show that the progress of carbonation was experimentally confirmed to obey the linear law concerning time under step response as the results of accelerated carbonation test (Fukushima et al. 1986, Fukushima et al. 1993). This new law under step response (or power law more generally in the ordinary atmospheric environment showing the exponentially increasing tendency of the concentration of atmospheric carbon dioxide (Special Report of the Government of U.S. A., 1981) is discussed as the new prediction/evaluation methods of the progress of carbonation of concrete modeling the tendency of the increase in the concentration of atmospheric carbon dioxide as one of the global environmental problems.

## 3 RESEARCH RESULTS

### 3.1 Evaluations of suppressive effects against carbonation of polymeric surface coatings

Suppressive effects of polymeric surface coatings under the accelerated condition of 30$^o$C, 60% RH, $CO_2$; 5% (in this case, the deterioration of polymeric surface coatings are not expected, because there is no UV light irradiation, and the temperature and humidity are mild and constant) accelerated carbonation test was done for about one year for test specimens of concrete treated with polymeric and other surface coatings inorganic shown in Table 1, and the results of evaluation of the progress of carbonation are shown in Table 2 and 3, and Fig.2.

Polymeric coating materials such as vinyl wallpaper, acrylic and epoxy resin show the greater suppressive effects against carbonation than other inorganic ones.

On the other hand, diffusion coefficients of $CO_2$ in various surface-finishing materials could be calculated by comparing theoretically obtained equation for two parameter A and B, and experimentally obtained values to. As shown in Table 3, diffusion coefficients become smaller in proportion as the suppressive effects are greater.

Except for untreated concrete specimens, for concrete specimens treated with surface finishing materials there are some induction time before the progress of carbonation starts, and the progress of carbonation obeys parabolic law involving a constant term (X=Avt – B) as follows:

$$X = 0 : 0 \le t \le t_0 = (\frac{B}{A})^2 \quad (1)$$

$$X = A\sqrt{t} - B \le t \quad (2)$$

**Table 1. Type of surface finishing materials**

| *Symbols* | *Type of polymeric coating* |
|---|---|
| a | no finish |
| b | vinyl wall paper |
| c | emulsion paint |
| d | synthetic resin plaster |
| e | acryl based sprayed finish |
| f | epoxy based sprayed finish |

**Table 2. Evaluation of the suppressive effects of surface finishing materials on Neutralization (carbonation) of concrete (Part 1 least-square method analysis)**

| | *Type of Specimen* | *Equation obtained by least-square method* | *Correlation coefficient* |
|---|---|---|---|
| Normal Concrete for Field-mixed Use | N-R-a | X = 0.65 + 14.45 vt | r = 0.980 |
| | N-R-b | X = -0.22 + 0.16 vt | r = 0.732 |
| | N-R-c | X = -1.27 + 9.43 vt | r = 0.992 |
| | N-R-d | X = -7.09 + 6.9 vt | r = 0.998 |
| | N-R-e | X = -3.96 + 7.06 vt | r = 0.959 |
| | N-R-f | X = 0.53 + 12.36vt | r = 0.994 |
| Concrete for precasting use | N-R-a | X = 4.78 + 8.3 vt | r = 0.956 |
| | N-R-c | X = -0.91 + 13.03 vt | r = 0.996 |

### 3.2 Equivalent Thickness of Cover Concrete

We can convert these suppressive effects of surface coatings against carbonation into equivalent thickness of cover concrete. The conversion methods are shown in Fig.6. For thickness of off-form concrete ($X_0$), the time when its carbonation becomes completed can For thickness of off-form concrete (X0), the time when it's carbonation becomes completed ($T_o$) can be given as follows:

$$T_0 = \left( \frac{A_0}{X_0} \right)^2 \qquad (3)$$

For concrete treated with surface coatings, the time becomes elongated up to $T_1$ due to their suppressive effects shown as follows:

$$T_1 = \left( \frac{A_0}{X_0 + B} \right)^2 \qquad (4)$$

**Table 3 Evaluation of the suppressive effects of surface Finishing materials on Neutralization of concrete (Part 2 calculation of effective diffusion coefficients of carbon dioxide)**

| *Types of Surface Finishing materials* | *Parameters* *A* (mm/month1/2) | *B* (mm) | *Surface equilibrium* *m* *constant* | *induction period* *t* *(month)* | *thickness* *L* *(mm)* | *diffusion coefficient* *D* *(mm2/month)* |
|---|---|---|---|---|---|---|
| a | 14.5 | - | 1 | - | - | 3.89 |
| b | - | - | - | - | - | - |
| c | 9.4 | 1.3 | 0.65 | 0.14 | 1 | 0.494 |
| d | 6.9 | 7.1 | 0.48 | 1.06 | 3 | 0.27 |
| e | 5.9 | 4.5 | 0.41 | 0.58 | 5 | 0.72 |
| f | 7.1 | 4.0 | 0.49 | 0.32 | 5 | 0.82 |

This actually means that thickness of cover concrete increase from $X_0$ to $X_0'$ shown as follows:

$$X_0' = \frac{A}{A_0} B + \frac{A}{A_0} X_0 \qquad (5)$$

$X_0'$ is the equivalent thickness corresponding to suppressive effects of surface coatings against carbonation. If we set this equivalent thickness in advance, the progress of carbonation of concrete treated with surface coatings really obeys parabolic law as that of carbonation of off-form concrete as follows:

$$X_0' = A_0 \sqrt{t} \qquad (6)$$

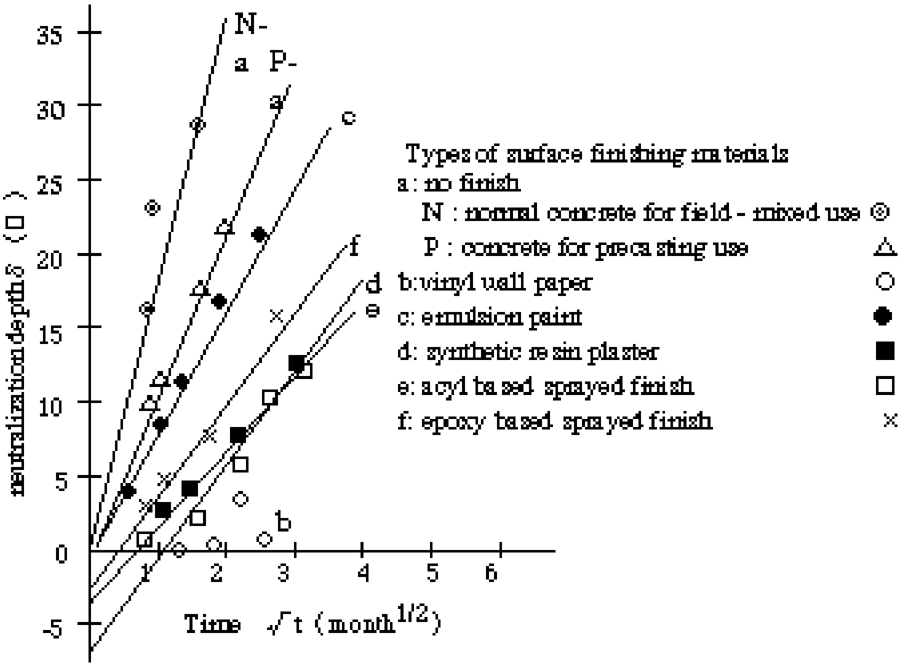


**Figure 2 Results of suppressive effects of polymeric coatings materials on the progress of carbonation of concrete by accelerated carbonation test (30°C, 60%R.H., CO2: 5%)**

### 3.3 New Prediction/Evaluation Methods of the Progress of Neutralization of Concrete by lLnear law and Power Law

By step response that the $CO_2$ concentration starts from 5%, and then to 10, 15, 20% constant value, by increasing by 5% step by step every month (under constant temperature constant humidity condition of 20 °C and 60%RH ), accelerated carbonation test was carried out for various types of concrete (3 types of normal Portland cement concrete, blast-furnace Portland cement concrete, fly ash Portland cement concrete of 40, 50, and 60% water cement ratios with and without acryl polymer type coating). Figure 4 and 5 show the test results. It is obvious that the progress of neutralization of concrete under the condition of step response obeys the linear law concerning time, on the contrary to the generally accepted parabolic law under the condition of constant $CO_2$ concentration, whether polymeric coating is made or not. From the consideration from theoretical analysis for the case of exponential increase of the concentration of atmospheric $CO_2$, power law is appropriate to be set of the concentration of atmospheric $CO_2$, because it shows the mediate law between the parabolic law in case of constant concentration of $CO_2$ and linear law under step response for the modified law for the progress of neutralization of concrete considering the increasing tendency.

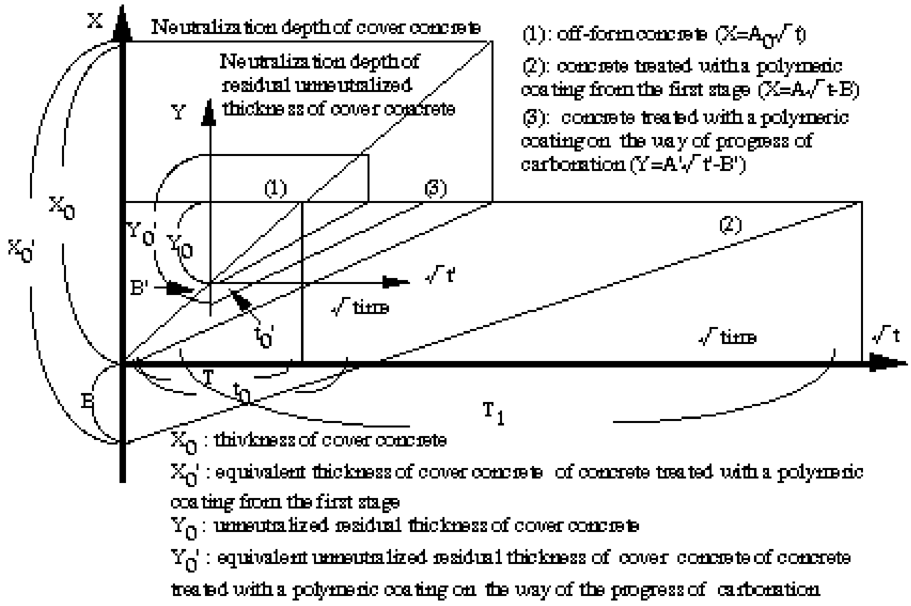


**Figure 3. Schematic diagram of the concept of equivalent thickness of cover concrete talking into account of the suppressive effects of polymeric coatings on the progress of carbonation of concrete**

### 3.4 Evaluation methods of suppressive effects of polymeric surface coatings against corrosion of reinforcing steel

If we can stop, retard or suppress the progress of corrosion of reinforcing steel by effective surface treatment with such polymeric surface coating materials when the corrosion of reinforcing steel is threatening to start as the progress of carbonation of cover concrete reaches very near reinforcing Steel, we can make use of the results in order to do the rational durability design for the best setting of physical service life of RC components and / or buildings by effective elongation.

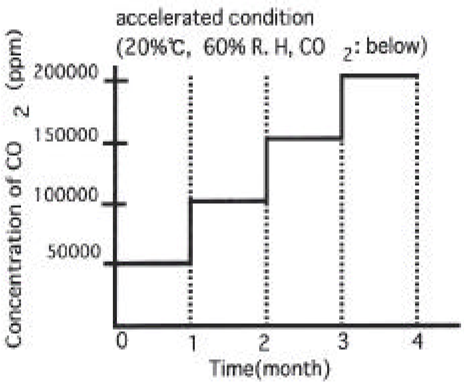


**Figure 4. Method of Accelerated Carbonation Test by Step Response Molding the Increasing Tendency of Atmospheric $CO_2$.**

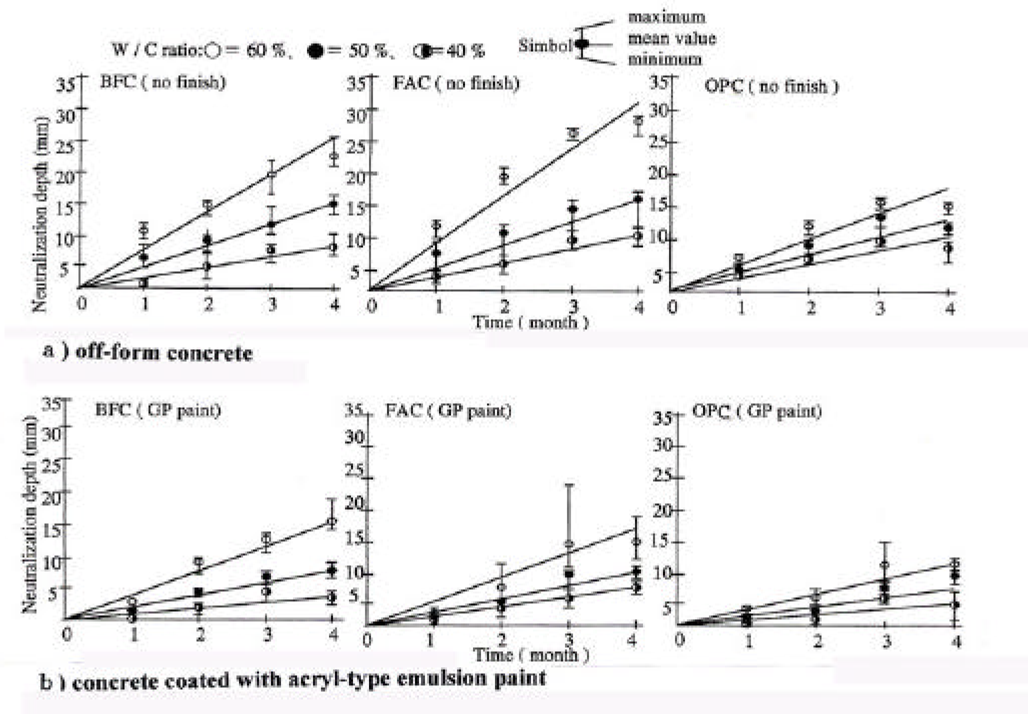


**Figure 5a) and5b). Experimental results of accelerated carbonation test by step response method for various types of concrete (4a) Off-form concrete; 4b) concrete coated with acryl resin-type GP paint**

In order to confirm experimentally if we can suppressive the progress of corrosion of reinforcing steel by polymeric surface coatings or not, we carried out accelerated corrosion test for 16 months, and outdoor and indoor exposure tests for 11 years for test sediments of reinforced concrete treated with such polymeric surface coatings after we confirm that carbonation of cover concrete reaches behind reinforcing steel by accelerated carbonation (20 $^{o}$C, 60%RH, CO2: 5%) for 5 months.

As the results, we could find that polymeric surface coating materials show, to a considerable extent, suppressive effects against the corrosion of reinforcing steel, as far as the severe deterioration of polymeric surface coatings themselves does not occur, and chloride ions are not included in concrete

**Table 5. Results of Evaluation of the progress of Corrosion under accelerated condition (50 °C, 95%) (Y : Rates of Corrosion Loss[%], t : time[month])**

| Type | Nomenclature | Evaluation law | Correlation coefficient |
|---|---|---|---|
| River Sand Specimens | N-R-a | Y = 0.44+0.26vt | r = 0.980 |
| | N-R-b | Y = 0.07+0.61vt | r = 0.976 |
| | N-R-c | Y = 0.45+0.02vt | - |
| | N-R-d | Y = 0.47 +0.59vt | r = 1.000 |
| | N-R-e | Y =-0.20+0.60vt | r =0. 498 |
| | N-R-f | Y =-0.47 + 0.63vt | r = 0.998 |
| | N-R-h | Y = -0.51 +0.64vt | r = 0.993 |
| | N-R-i | Y =-0.44+0.83vt | r =0. 999 |
| | N-R-j | Y =-0.09 + 0.50vt | r = 0.933 |
| | P-R-a | Y =0.09 + 0.50vt | r = 0.999 |
| | P-R-b | Y =-0.09+ 0.36vt | r = 0.693 |
| | P-R-c | Y =0.43+ 0.10vt | r = 0.501 |
| | P-R-e | Y =-0.23 + 0.61vt | r = 0.425 |
| Sea Sand Specimens | N-S-a | Y =-0.20 + 0.41vt | r = 0.905 |
| | N-S-b | Y = 0.38 + 0.33vt | r = 0.700 |
| | N-S-c | Y =-0.18 + 0.33v t | r = 0.933 |
| | N-S-e | Y = 0.16 + 0.40vt | r = 0.822 |
| | P-S-a | Y =-0.22 + 0.41vt | r = 0.834 |
| | P-S-b | Y = 0.16 + 0.54vt | r = 0.953 |
| | P-S-c | Y =-0.09+ 0.44vt | r = 0.972 |
| | P-S-e | Y = 0.32 + 0.54vt | r = 0.949 |

above a certain allowed concentration, by the shielding function against the diffusion of water vapour and oxygen in neutralized concrete.

1) On the other hand, we evaluated the progress of corrosion of reinforcing steel by applying the least-square method for experimental data. The results are summarized in Table5 and 6. We could summarize as follows: In case of no inclusion of chloride ions above a certain allowed concentration.

The suppressive effects against corrosion of polymeric surface coatings can be evaluated by two parameters $A_1$, $B_1$ of the parabolic law involving a minus constant term concerning time by considering the induction period $t_{01}$ as follows:

$$Y = 0 : 0 \leq t \leq t_{01} \equiv \left( \frac{A_1}{B_1} \right)^2 \qquad (7)$$

$$Y = A_1 \sqrt{t} - B_1 \qquad t_{01} \leq t \qquad (8)$$

Here, Y is the ratio of corrosion loss of reinforcing steel [%].

2) In case of inclusion of chronicle ions above certain allowed concentration. Any polymeric surface coating materials do not show effective suppressive effects against corrosion. The progress of corrosion of reinforcing steel can be evaluated by two parameters $A_2$, $B_2$ of the parabolic law involving a plus constant term.

$$Y = A_2 \sqrt{t} + B_2 \qquad (9)$$

Y = $A_2$ vt + $B_2$

This means that even if the carbonation never reaches behind reinforcing steel, the corrosion progresses continuously due to the existence of chloride ions. Polymeric surface coating materials such as emulsion paint, synthetic resin plaster, acryl and epoxy resin based sprayed finishes show these suppressive effects.

**Table 6. Results of Evaluation of the Progress of Corrosion under outdoor exposure (not sheltered from rain) (Y: Rate of Corrosion Loss [%],t : Time table 6[Year])**

| Type | Nomenclature | Evaluation law | Correlation coefficient |
|---|---|---|---|
| River Sand | N-R-a | Y = 0.690+0.217vt | r = 0.937 |
| | N-R-b | Y = 0.502+0.205vt | r = 0.854 |
| | N-R-e | Y = -0.124+0.348vt | r = 0.76 |
| | N-R-f | Y = 0.475 +0.271vt | r = 0.843 |
| | N-R-h | Y =-0.030+0.454vt | r = 0.873 |
| | N-R-i | Y =-0.057 + 0.439vt | r =0. 837 |
| | P-R-e | Y =-0.155 + 0.405vt | r = 0.890 |
| | P-R-a | Y =0.330 + 0.188vt | r = 0.963 |
| Sea Sand | N-S-a | Y = 0.035 + 0.142vt | r = 0.726 |
| | N-S-e | Y =-0.468 + 0.813vt | r = 0.964 |

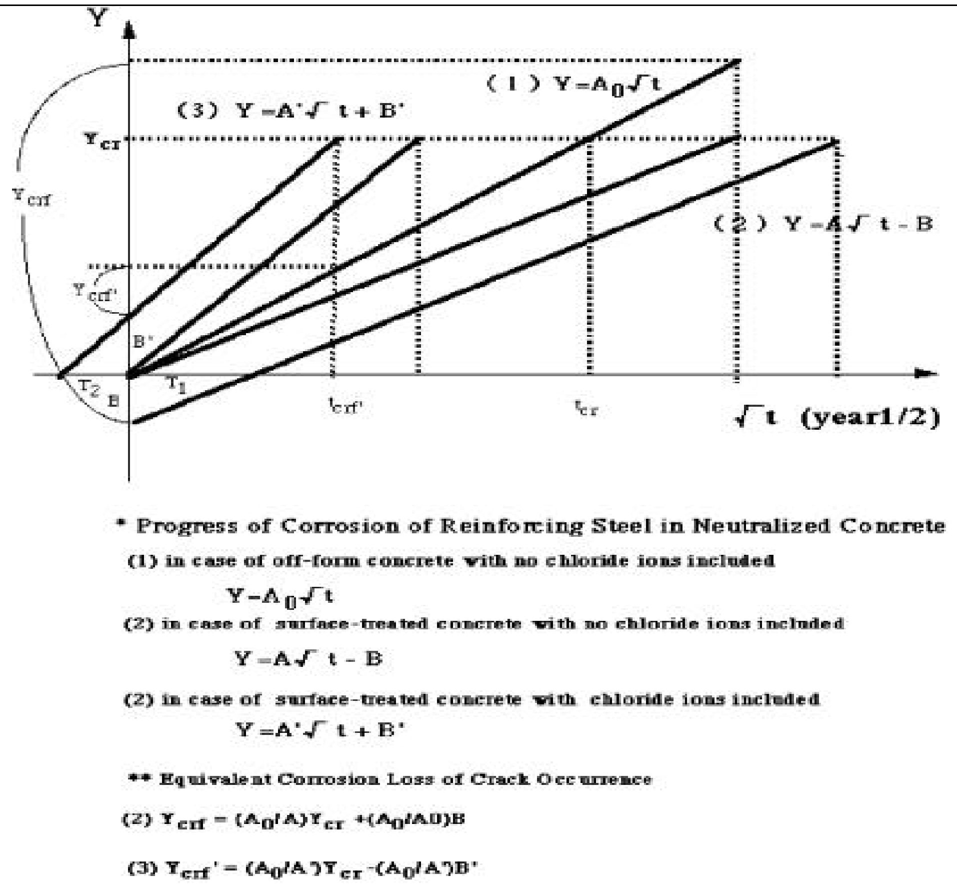


**Figure 6. Concrete of Conversion of Suppressive Effects against Corrosion of Surface Finishing Materials into Equivalent Corrosion Losses of Crack occurrence**

### 3.5 Equivalent corrosion loss of crack occurrence

As the analogy to the conversion of suppressive effects against carbonation of polymeric surface coating materials into equivalent thickness of cover concrete, we can convert suppressive effects against corrosion into equivalent corrosion loss of crack occurrence in cover concrete.

This concept is illustrated in Fig.6. If we set in advance these equivalent corrosion loss of crack occurrence, the progress of corrosion of reinforcing steel in neutralized concrete with surface treatment can be discussed in the same way as corrosion in off-form neutralized concrete without surface treatment.

### 3.6 Setting methods of physical service life of RC with polymeric surface coatings

Figure 7.shows the diagram of this concept**.** Physical service life of off-form RC components and / or buildings under ordinary atmospheric environment in the mild climate can be set by the progress of carbonation and corrosion as T1 + T2. If we taking into account the increasing tendency of atmospheric $CO_{2,}$ we should adopt the power law concerning time instead of parabolic law for the progress of neutralization (carbonation) of concrete.

If we make effective use of surface finishing materials, however, the service life of reinforced concrete buildings can be prolonged by suppressive effects of surface finishing materials against the progress of carbonation of concrete and corrosion of reinforcing steel under the assumption that surface finishing materials have high durability for long time, not deteriorating so much as in the case of fluoropolymer, or that even if they deteriorate, the periodic exchange or renewal of them is done. However, we should also consider life cycle design, taking into account ecobalance performance such as the easiness of demotion and recyclabilly after designed service life in order to establish sustainable construction considering environmental harmony.

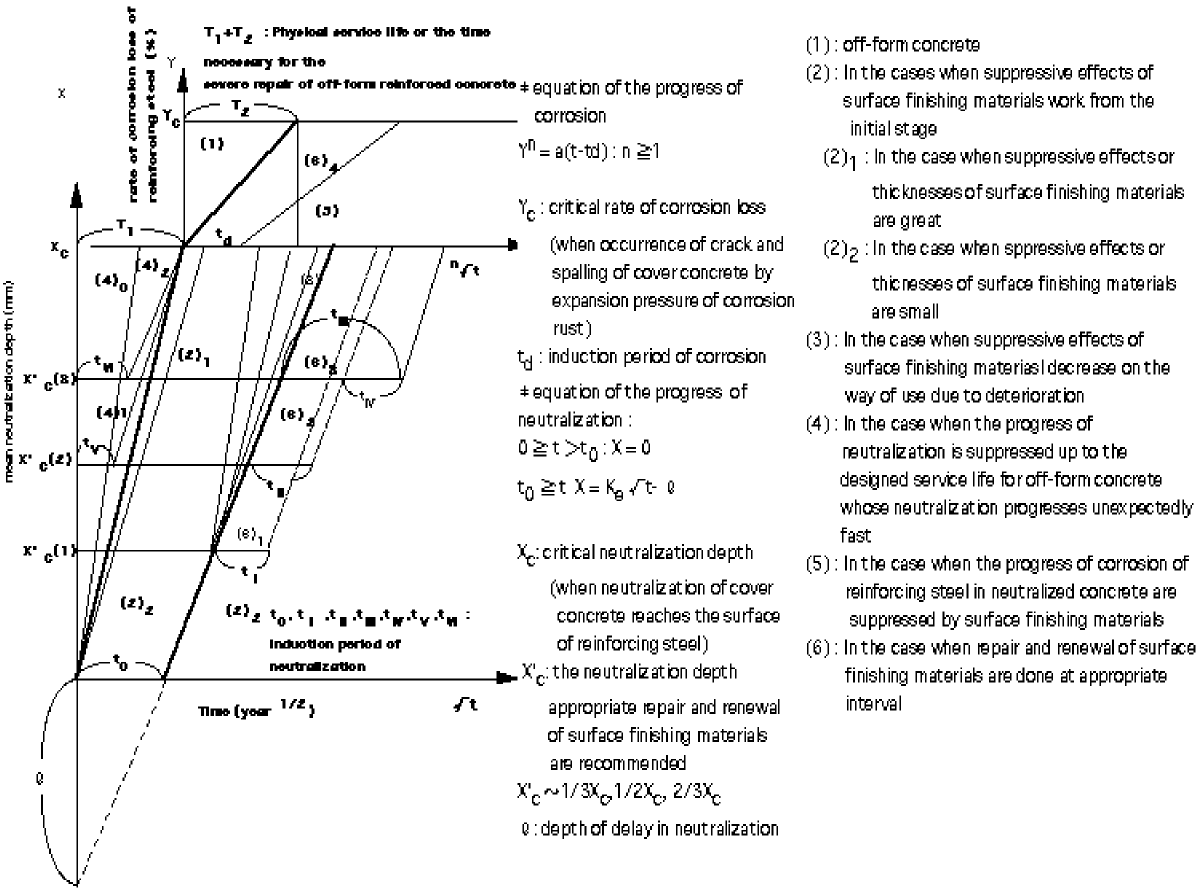


**Figure 7. Concept of Setting the Physical Service Life of Reinforced Concrete Considering the Suppressive Effects of Surface Finishing Materials**

## 4 CONCLUSIONS

1) Polymeric surface coating materials are said to have rather shorter service life of about 10 years under outdoor conditions due to weathering. However, if periodic renewal or repaint are carried out, they will greatly contribute to prolong the physical serried life of RC components and / or buildings due to their suppressive effects against carbonation of concrete are corrosion of reinforcing steel.

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2) The progress of neutralization (carbonation) of concrete under the condition of step response obeys the linear law concerning time, on the contrary to the generally accepted parabolic law under the condition of constant $CO_2$ concentration, whether polymeric coating is made or not. From the consideration from theoretical analysis for the case of exponential increase of the concentration of atmospheric $CO_{2,}$ power law is appropriate to be set for the modified law for the progress of concrete considering the increasing tendency.

3) We should establish eco-life-cycle design of building structural composite components such as reinforced concrete (RC) and continuous fiber reinforced concrete (FRPRC), paying attention to the harmony of long service life with eco-balance performance for sustainable construction in the twenty-first century.

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# The Effect Of Algae On Mineral Powder Dissolution Rates

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**Summary: Experiments have been performed to study the release of cations into an aqueous medium from a variety of minerals commonly found as constituents in stone masonry in the presence of pure algal cultures. The aim of the experiment is determine the influence of algae on the dissolution of rock-forming minerals, leading ultimately to a model of rock weathering that incorporates the chemical action of micro-algae. The minerals used were albite feldspar, muscovite mica, quartz and the carbonates siderite and calcite. Powdered samples of each of these minerals immersed in distilled water were exposed to axenic cultures of the green algae *Chlorococcum tetrasporum* and *Scenedesmus obliquus* for four weeks. In order to monitor uptake of chemical components of the minerals into solution, the water was sampled weekly and analysed for pH and chemical composition. Comparison with a system of control experiments allowed us to distinguish the influence of algae from non-organic dissolution.**

**Preliminary results from this experiment show that the yields of aqueous cations are much larger for carbonates than silicates, both in the presence and absence of algae. The algae create a mildly alkaline environment, which may influence the mineral dissolution for certain minerals. Algae are seen to enhance Fe dissolution from the carbonates, which may contribute to the development of iron rich patinae on building stone. The algae appear to enhance Ca release from calcite, an effect linked to the pH change mediated by the algae. Finally the algae appear to have no direct influence on feldspar dissolution, however the maintenance of a humid environment by a biofilm at the masonry surface will lead to the destruction of feldspars and possibly precipitation of pore-blocking clays or Al-hydroxides.**

**Keywords: Biodeterioration, algae, minerals, silicates, carbonates.**

## 1 INTRODUCTION

The degradation of natural stone masonry is a highly complex system influenced by a large number of environmental factors. One of the key factors in biodegradation is biological weathering, in which organisms act to destroy or weaken the mineral constituents of the stone. Phototrophic organisms are among the primary colonisers of building surfaces (Wakefield *et al.*, 1996), and thus are some of the first organisms to interact with the stone. Algae as a phototrophic organism is mostly recognised for its role as a nutrient source for other organisms such as fungi and bacteria which have been shown experimentally to be more damaging to stone (Ortega-Calvo *et al*. 1993; Mishra *et al.* 1995; Warscheid & Braams, 2000). Until recently it was believed that algae were likely to cause no more damage to a building stone substrate than surface staining (Pietrini *et al*, 1985). However, recent studies have shown that chemical and physical damage caused by algae include the exudation of corrosive organic acids (Caneva *et al.* 1992*)*, the uptake of Ca into the microbial cells (Viles & Moses, 1996) and the enlargement of pore sizes and spaces due to the penetration of hyphae cellular filaments (Arino & Saiz-Jimenez, 1996). This experiment is unique in that it exposes both carbonate and silicate minerals to pure algal cultures as opposed to a single exudate such as oxalic acids (Welch & Ullman, 1993). Using mineral powders to increase the chemical yield and and ICP-AES to resolve concentrations to below 1mg/l this research can show effects that have previously been irresolvable to the instrumentation available.

The approach was to examine, under controlled conditions, the influence of algae on dissolution of the individual mineral constituents of common silicate-based or silicate + carbonate-based building stones, especially sandstones. The core objective is to better understand the biological and chemical processes taking place at the interface between mineral and organism This should ultimately enable us to identify which minerals are vulnerable to algal attack, and hence to construct a model of algal biodeterioration based upon an understanding of the response of different mineral microfabric elements present in natural stone materials. Due to the relatively slow reaction rates and limitations on experimental run times set by the longevity of the algal culture, the minerals studied were powdered to provide a large contact surface area with the algal biofilm, thus enhancing yields of dissolved cations. We also used pure algal cultures so as to distinguish clearly the influence of algae from microbial communities containing both algae and other microbes, such as bacteria and fungi, already known for their aggressive activity (Bock & Sand, 1993). The chemical composition of the aqueous medium was monitored during the course of the experiment in order to detect the presence of cations released into the water from the mineral surfaces, either directly or via the algal biofilm.

## 2 METHODOLOGY

The minerals used in this experiment were the tectosilicate feldspars albite, labradorite and orthoclase, quartz, the potassic phyllosilicate muscovite, and the carbonates calcite, dolomite and siderite (Supplied by Gregory, Bottley and Lloyd, 13 Seagrove Road, London, SW6 1RP, source unknown). Chemical composition of each mineral was determined semi-quantitatively by energy dispersive electron probe micro-analysis of polished grain mounts on a Hitachi S4100 cold field emission scanning electron microscope (SEM).

Mineral samples were crushed to powder in an agate pestle and mortar then sieved through a 450µm nylon, one use only sieve. A 2g split of each powder sample was then placed in a 25ml universal bottle containing 20ml of distilled water. The flasks were then sealed and autoclaved to ensure sterility.

The algae used in this experiment were a mix of three axenic algal cultures *Chlorella vulgaris* (CCAP 211/11P), *Scenedesmus obliquus* (CCAP 276/7) and *Stichococcus bacillaris* (CCAP 379/1D) purchased from CCAP (Culture Collection of Algae and Protozoa, Cumbria, U.K.). The algae were cultured using Bolds Basal Medium (BBM) in a biological incubator on an 18/6 hr light/dark cycle with a temperature of 16 ± 2 $^{o}$C. To avoid chemical contamination of the water in the experimental flasks by the BBM, the algae were washed using a method adapted from Yee *et al* (2000). 100ml of the three algal species was removed from the incubator and mixed with the other species in a 500ml conical flask on an orbital shaker for ten minutes. The 300ml of algal mix was then divided into 12x25ml universal bottles. The bottles were centrifuged for 10 minutes at 3200 rpm, to separate the nutrient media from the algae, with the supernatant being poured away after the centrifugation. 20ml of Autoclaved Distilled Water (ADW) was then added into each flask and mixed for ten minutes on the orbital shaker. 0.001M EDTA was then added to the algae and left for sixty minutes, centrifuged and the supernatant poured away again. 20ml of ADW was added to the algae and left to soak the algae overnight. This was centrifuged off the next day and 20ml of ADW was added to the algae remaining in the flask. 2ml of this mixture were then transferred into the bottles containing the water and mineral powders. Some bottles were left free of algae in order to determine the solubility of mineral powders in presence of water alone. Other bottles contained only distilled water and algae to determine any cationic output from the algae in the absence of a mineral powder. Finally, some bottles were filled only with distilled water to act as analytical blanks. Each type of experimental charge was triplicated.

The flasks were kept in a Gallenkamp biological incubator on an 18/6 hr light-dark cycle at a temperature of 16 ± 2 $^{o}$C and sampled every seven days for pH using a Piccolo Plus ATC pH/C meter with a HI1295 Amplified Electrode fitted (Hanna Instruments, Woonsocket, USA). Further water samples were pipetted off for chemical analysis at the same time. Note that a batch of experimental charges were made up at the outset for each water sample extraction so as to avoid problems of progressive loss of aqueous medium through successive sampling extractions, and to minimise biological contamination.

Concentrations of aqueous Al, Ca, Fe, K, Mg, Na, P were measured using inductively coupled plasma atomic emission spectrometry (ICP-AES) Perkin Elmer Optima 3000 using appropriate multi element standards and blank subtraction, ultra high purity water (18MΩ Elga Stat UHP) was used for all dilutions. These elements were selected as they are important components of either the minerals or the algae. All water samples in one batch were tested in a randomised sample order to avoid the effects of machine drift. Due to the quartz torch utilised in this instrument, the measurements contain a large Si analytical blank making analysis of this important element impractical. Standard deviations are not shown in the results section of the analysis due to space constraints, also in no instance in this investigation does the standard deviation affect the trendline of an element to a significant degree.

## 3 RESULTS

Chemical compositions for the minerals used in the experiment are typical for their types (see Deer, Howie & Zussman, 1992). Albite (sodium aluminium silicate) contained small concentrations of potassium, and siderite (iron carbonate) contained a few percent each of Mg and Mn. Calcite (calcium carbonate) contained no detectable Mg, Fe or Mn.

The pH data (Table 1) shows that the algae has an effect on the acidity/alkalinity levels in the bottles over the course of the experiment.

**Table 1. Variation of pH in the aqueous medium over time**
**All 1s errors £ 0.2 pH units**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* | *Day 28* |
|---|---|---|---|---|---|
| WATER ONLY | 6.8 | 6.7 | 6.9 | 6.7 | 6.8 |
| WATER AND ALGAE | 6.7 | 6.7 | 6.6 | 6.8 | 6.9 |
| ALBITE AND WATER | 6.8 | 7 | 6.9 | 6.8 | 7.1 |
| ALBITE AND ALGAE | 6.9 | 6.7 | 6.9 | 7.1 | 7.4 |
| QUARTZ AND WATER | 6.9 | 6.8 | 6.8 | 7.0 | 6.9 |
| QUARTZ AND ALGAE | 6.7 | 6.6 | 6.7 | 6.8 | 7.1 |
| MUSCOVITE AND WATER | 6.9 | 7.2 | 7 | 6.9 | 6.6 |
| MUSCOVITE AND ALGAE | 6.9 | 6.9 | 7.0 | 7.2 | 7.4 |
| CALCITE AND WATER | 7 | 6.8 | 7 | 6.8 | 7.3 |
| CALCITE AND ALGAE | 6.8 | 6.9 | 7.5 | 7.2 | 7.3 |
| SIDERITE AND WATER | 6.8 | 7.1 | 7.1 | 6.8 | 6.7 |
| SIDERITE AND ALGAE | 7.1 | 6.9 | 6.7 | 6.9 | 7.3 |

In the first seven days very little difference can be discerned between bottles containing algae and those without. However after 7 and 14 days a noticeable rise in pH of those flasks containing algae can be distinguished for all minerals. Muscovite with algae deviates most from the water and mineral flask, leading to a difference of 0.8 pH units by the end of 28 days, due to both a decrease in the bottle lacking algae and an increase in the bottle containing algae. The bottle containing quartz with algae showed very little deviation from that without algae. Calcite with algae shows a more rapid rise in pH than calcite alone, but values for both converge on the same value, higher than the starting value.

The study of pH in this environment is important to as the dissolution rates of minerals can change rapidly over small changes in the surrounding pH. Stumm & Morgan (1996), show that calcite provides particularly strong evidence of this in the pH range being studied, falling from a dissolution rate of $10^{-2}$ Mol $m^{-2}h^{-1}$ to $10^{-4}$ Mol $m^{-2}h^{-1}$ between the pH's of 5 and 10.5 Dolomite is slightly below this level of dissolution, whilst the other silicate and oxide minerals display much lower dissolution rates, for example, $10^{-8}$ Mol $m^{-2}h^{-1}$ for albite and $10^{-9}$ Mol $m^{-2}h^{-1}$ for muscovite and quartz at pH 7.

Element concentrations in the aqueous medium measured at seven day intervals are given in Tables 2-7. Concentrations in experimental charges containing only water and algae generally showed little change throughout the experiment (Table 2 and Figure 1). Detectable amounts of Mg were present in the water+algae sample. This could be due to dead algae in the bottle decomposing and releasing the Mg-bearing porphyrin ring component of chlorophyll. Ca, K, Na and P concentrations were above the water blank level in the water+algae samples, but K, Na and P decreased over time. The latter three may have been contamination from the BBM left over after washing of the algae, and were possibly used as nutrients by the algae over time. Ca shows a peak at 14 days, and may have been released by the algae. Levels of Al, Fe and Mn in the water+algae charges were all below the detection limits of the instrument. The cause of the high water blank value for Ca at 7 days is unknown.

**Table 2. Elemental concentrations ($mgl^{-1}$) in water+algae samples and the water only experimental blanks (italicised) over time. Detection limits (right hand column) were quantified by external calibration using multi element standards (Leemer Labs UK).**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* | *Detection limit* $mgl^{-1}$ |
|---|---|---|---|---|---|
| Al | 0.007 (*0.004*) | 0.002 (*0.032*) | 0.041 (*0.001*) | 0.0291 (*0.002*) | 0.044 |
| Ca | 0.607 (*0.055*) | 0.400 (*0.547*) | 1.074 (*0.045*) | 0.4465 (*0.150*) | 0.041 |
| Fe | 0.002 (*0.001*) | 0.001 (*0.000*) | 0.003 (*0.000*) | 0.0119 (*0.000*) | 0.024 |
| K | 1.393 (*0.332*) | 0.840 (*0.271*) | 0.421(*0.472*) | 0.3454 (*0.363*) | 0.100 |
| Mg | 0.158 (*0.011*) | 0.106 (*0.064*) | 0.139 (0.*008*) | 0.0967 (*0.023*) | 0.027 |
| Mn | 0.000 (*0.000*) | 0.024 (*0.001*) | 0.003 (*0.000*) | 0.000 (*0.000*) | 0.024 |
| Na | 4.642 (*1.389*) | 4.252 (*2.859*) | 4.130 (*1.215*) | 1.357(*1.009*) | 0.033 |
| P | 0.797 (*0.093*) | 0.134 (*0.166*) | 0.024 (*0.125*) | 0.018 (*0.193*) | 0.069 |

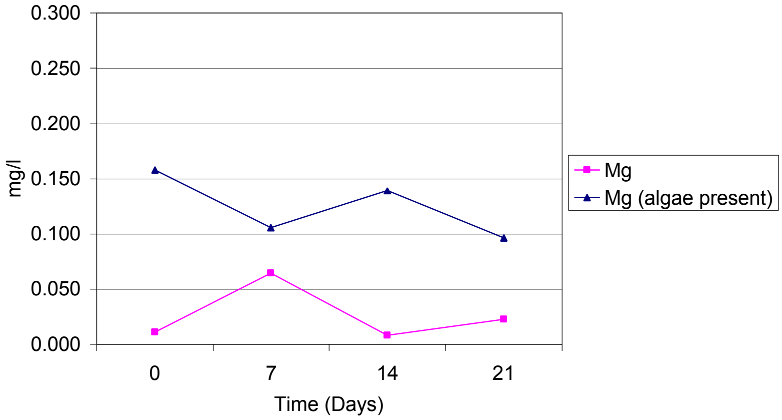


**Fig. 1. Mg concentrations through time in the water only and water+algae samples**

The chemical trends in **calcite-**bearing charges (Table 3 and Figure 3) show certain pertinent differences when compared to the water only control experiment (Table 2 and Figure 1). The release of Ca into the water from the calcite was very pronounced over this time period. Without algae present the concentrations were around $8mgl^{-1}$, compared to below $1mgl^{-1}$ for water only. When algae was present the concentration of Ca rose to 30mg/l.

**Table 3. Elemental concentrations (mg/l) in aqueous medium containing calcite and algae over time. Figures in brackets are for water and calcite with no algae. Detection limits as for Table 2.**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* |
|---|---|---|---|---|
| Al | 0.037 (*0.057*) | 0.030 (*0.060*) | 0.197 (*0.093*) | 0.086 (*0.058*) |
| Ca | 14.568 (*8.219*) | 28.805 (*7.882*) | 8.544 (*7.391*) | 16.967 (*17.423*) |
| Fe | 0.000 (*0.002*) | 0.001 (*0.000*) | 0.004 (*0.001*) | 0.005 (*0.003*) |
| K | 5.828 (*0.860*) | 0.586 (*0.591*) | 1.046 (*0.436*) | 1.154 (*0.520*) |
| Mg | 0.301(*0.133*) | 0.394 (*0.167*) | 0.583 (*0.158*) | 0.419 (*0.127*) |
| Mn | 0.014 (*0.002*) | 0.040 (*0.003*) | 0.000 (*0.013*) | 0.012 (*0.003*) |
| Na | 18.363 (*4.611*) | 2.967 (*5.572*) | 4.984 (*6.504*) | 2.953 (*3.332*) |
| P | 0.843 (*0.061*) | 0.010 (*0.128*) | 0.258 (*0.025*) | 0.079 (*0.000*) |

Note how the calcite dissolution rate increases as pH falls. A reciprocal relationship of the pH (table 1) and Ca concentrations (table 3) would have been expected in these experiments on this basis, and indeed a distinct drop in Ca at day 14 coincided to a significant increase in pH at the same time.

The calcite lacked detectable Mg, but once again aqueous Mg concentrations were higher for the calcite+algae sample than the calcite alone, presumably due to chlorophyll release.

**Siderite** is an iron carbonate, commonly found as a cementing fabric element in sandstones. Aqueous Fe shows a strong rise in the siderite+algae sample but no significant change in the sample lacking algae (Table 4 and Fig 3). The implication of this is that the algae were enhancing removal of Fe cations from the mineral surfaces. Mg and Mn also show interesting trends; Mg concentrations for siderite alone are up to $14mgl^{-1}$, whilst when algae are present the Mg concentrations are *lower*. The same happens with Mn, with aqueous concentrations for siderite alone being around $4mgl^{-1}$, falling with algae present to below detection limits. There are two possible explanations for this effect. First, Mn ions were released into the aqueous medium due to inorganic reactions (hydrolysis), and the algae are using the aqueous Mg and Mn as a nutrient source. Secondly the algae may have been acting on the mineral surfaces to reduce the solubility of the Mg and Mn complexes.

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**Table 4. Elemental concentrations (mgl$^{-1}$) in water+siderite and water with siderite+algae samples over time. Figures in brackets are water+siderite only. Detection limits as for Table 2.**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* |
|---|---|---|---|---|
| Al | 0.002 (*0.000*) | 0.059 (*0.000*) | 0.167 (*0.011*) | 0.068 (*0.019*) |
| Ca | 1.771 (*2.172*) | 2.097 (*1.701*) | 1.789 (*2.156*) | 1.653 (*1.786*) |
| Fe | 0.000 (*0.004*) | 0.014 (*0.001*) | 0.042 (*0.002*) | 0.031 (*0.009*) |
| K | 0.874 (*1.091*) | 1.585 (*1.149*) | 0.961 (*0.744*) | 0.725 (*0.900*) |
| Mg | 6.585 (*6.952*) | 8.798 (*14.050*) | 6.856 (*8.541*) | 7.043 (*11.584*) |
| Mn | 0.890 (*0.855*) | 1.193 (*2.872*) | 0.009 (*1.106*) | 0.070 (*2.496*) |
| Na | 2.720 (*4.857*) | 6.475 (*5.636*) | 3.211 (*6.157*) | 2.784 (*3.632*) |
| P | 0.042 (*0.030*) | 0.193 (*0.211*) | 0.334 (*0.015*) | 0.026 (*0.000*) |

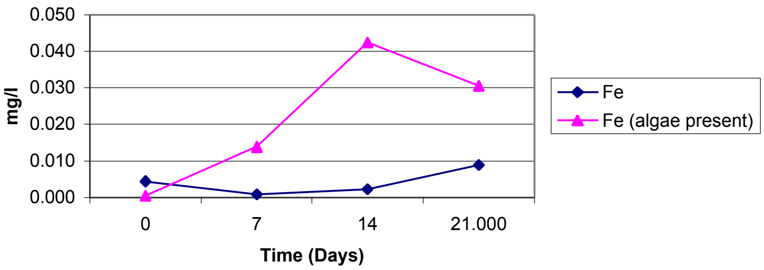


**Fig. 3. Fe concentrations through time in the water+siderite and the water with siderite+algae bottle**

**Quartz**, a silicon dioxide tectosilicate, is generally regarded as the most stable of the major rock forming minerals (Goldich, 1938). As the ICP-AES did not allow analysis for Si, it is impossible to know if there was any Si dissolution. Mg is significantly higher in the quartz+algae samples (Table 5), again probably due to chlorophyll release. Slightly enhanced levels of Ca, Na and P in the quartz+algae samples may be due to contamination from BBM.

**Table 5. Elemental concentrations (mgl$^{-1}$) for quartz+algae in aqueous medium over time. Figures in brackets are for quartz only. Detection limits as for Table 2.**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* |
|---|---|---|---|---|
| Al | 0.014 (*0.051*) | 0.018 (*0.116*) | 0.268 (*0.012*) | 0.259 (*0.105*) |
| Ca | 0.608 (*0.329*) | 2.600 (*0.856*) | 1.300 (*0.708*) | 1.646 (*0.401*) |
| Fe | 0.001 (*0.001*) | 0.005 (*0.000*) | 0.003 (*0.002*) | 0.030 (*0.004*) |
| K | 1.410 (*1.904*) | 2.790 (*1.427*) | 1.431 (*1.112*) | 1.300 (*0.810*) |
| Mg | 0.258 (*0.125*) | 0.654 (*0.306*) | 0.472 (*0.370*) | 0.431 (*0.173*) |
| Mn | 0.000 (*0.000*) | 0.022 (*0.000*) | 0.000 (*0.004*) | 0.000 (*0.001*) |
| Na | 2.695 (*4.644*) | 6.541 (*4.332*) | 3.488 (*1.395*) | 5.340 (*2.474*) |
| P | 0.235 (*0.203*) | 0.296 (*0.059*) | 0.140 (*0.028*) | 0.063 (*0.000*) |

**Albite** is a sodium aluminium tectosilicate. Mineral samples used here contained minor amounts of Ca and K. Aqueous Al showed a parallel rise both for samples with algae and those without (Table 6 and fig. 4). This indicates that the Al complex of albite was soluble in the water, rising to 0.9mg/l over 21 days, but was not affected or metabolised by the algae. Aqueous Na concentrations are significantly above blank levels in albite samples lacking algae. Similar levels are found early in the experiment in samples with algae, but later these decrease. Hence Na may have been dissolved out of the albite by water without the influence of algae (hydrolysis), but algae may have subsequently used it as a nutrient. Once again the Mg is higher in concentration when the algae is present

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**Table 6. Elemental concentrations ($mgl^{-1}$) for aqueous medium with albite+algae over time. Figures in brackets are albite only**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* |
|---|---|---|---|---|
| Al | 0.055 (*0.059*) | 0.032 (*0.055*) | 0.622 (*0.680*) | 0.900 (*0.789*) |
| Ca | 0.455 (*0.597*) | 0.453 (*0.178*) | 0.430 (*0.094*) | 0.357 (*0.217*) |
| Fe | 0.002 (*0.004*) | 0.002 (*0.003*) | 0.011 (*0.013*) | 0.091 (*0.018*) |
| K | 4.658 (*4.816*) | 5.469 (*3.323*) | 3.769 (*4.998*) | 3.956 (*5.418*) |
| Mg | 0.435 (*0.100*) | 0.163 (*0.057*) | 0.079 (*0.042*) | 0.148 (*0.083*) |
| Mn | 0.042 (*0.001*) | 0.008 (*0.002*) | 0.000 (*0.001*) | 0.003 (*0.000*) |
| Na | 4.208 (*4.287*) | 4.357(*4.773*) | 3.349 (*3.588*) | 3.470 (*5.961*) |
| P | 0.385 (*0.165*) | 0.419 (*0.054*) | 0.129 (*0.319*) | 0.083 (*0.139*) |

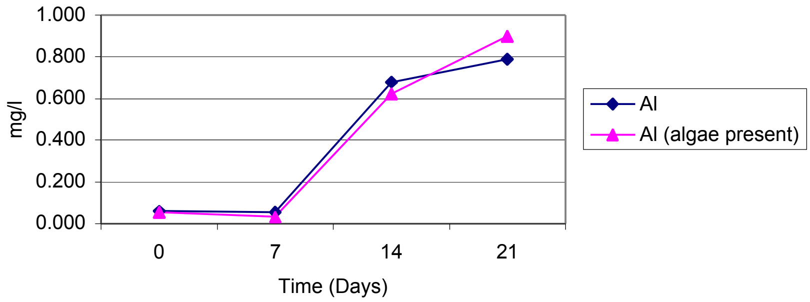


**Fig. 4. Al concentrations in aqueous medium through time for albite+algae and albite alone**

The muscovite elemental trends showed a small rise in Al when the algae was present, which may demonstrate a small amount of increased solubility under the influence of algae. All K values are well above blank levels. In addition, K shows a high initial concentration in the muscovite+algae sample but little change for muscovite alone, indicating a rapid inorganic release of K into solution that is either inhibited or rapidly metabolised by the algae. Ca is absent in muscovite, but aqueous Ca shows a peak at 14 days with algae present. This suggests that the algae released Ca as an extra-cellular product.

**Table 7. Elemental concentrations (mg/l) for aqueous medium with muscovite+algae over time. Figures in brackets are for muscovite only**

| | *Day 0* | *Day 7* | *Day 14* | *Day 21* |
|---|---|---|---|---|
| Al | 0.119 (*0.010*) | 0.073 (*0.092*) | 0.865 (*0.388*) | 0.228 (*0.056*) |
| Ca | 0.000 (*0.260*) | 1.057 (*0.243*) | 4.911 (*0.471*) | 1.219 (*0.836*) |
| Fe | 0.018 (*0.003*) | 0.005 (*0.002*) | 0.019 (*0.033*) | 0.014 (*0.002*) |
| K | 5.365 (*15.344*) | 5.942 (*6.441*) | 7.237(*5.564*) | 2.972 (*5.855*) |
| Mg | 0.054 (*0.031*) | 0.414 (*0.071*) | 0.167 (*0.068*) | 0.056 (*0.158*) |
| Mn | 0.003 (*0.000*) | 0.027 (*0.000*) | 0.002 (*0.001*) | 0.002 (*0.001*) |
| Na | 4.289 (*3.939*) | 6.092 (*3.162*) | 4.792 (*5.484*) | 3.824 (*3.576*) |
| P | 0.083 (*13.554*) | 1.489 (*0.252*) | 0.054 (*0.009*) | 0.002 (*0.307*) |

## 4 DISCUSSION

Each of the minerals investigated showed a distinctive set of chemical trends for the aqueous medium in which it was immersed. Algae appear to have created small but significant increases in pH. pH also increased when mineral powders were immersed in water in the absence of algae, with the exception of the rather unreactive quartz. However, alkalinity was usually further enhanced in the presence of algae, particularly for carbonates (as might be expected for these basic compounds) and the sodic feldspar albite. For most of the minerals there is no clear relationship between pH and aqueous cation concentration, but for calcite there is a clear reciprocal relationship with Ca. Stumm & Morgan (1996) have shown that dissolution rate of calcite decreases with increasing pH (fig. 2). This is consistent with the trend observed for calcite, but only if Ca was being partially absorbed by the algal biofilm once it had been released from the mineral surfaces. Hence the decrease in aqueous Ca observed at day 14 would have been due to a slower Ca release rate compared with its rate of uptake into the algae. Cation concentrations were much lower for the silicates, consistent with their much slower dissolution rates (Stumm & Morgan, op. cit.) as shown in fig. 2.

The experiment on siderite powder showed that algae are capable of mobilising Fe into aqueous solution. Siderite is a common cementing mineral in sandstones. Mobilisation and re-precipitation of iron is an important process in development of weathering patinae in sandstones (Bluck & Porter, 1991). Our results here indicate that algae may play an important role in this process. As with calcite, dissolution of siderite may destroy a cementing microfabric element of the stone, hence encouraging grain loss from the masonry surface. In contrast, algae appear to have either inhibited the release of Mn and Mg from siderite, or absorbed these cations into the biofilm immediately upon release. The latter alternative seems most likely as Mn and Mg tend to behave geochemically in a similar way to Fe. Further work will be required to distinguish between these two mechanisms.

Al is an important framework-building cation in feldspars. Al release patterns were identical in experimental charges with and without algae, suggesting that algae do not influence Al dissolution from feldspars. Feldspars are important constituents of sandstones. Al dissolution is important in weathering and weakening feldspar grains and precipitation of Al-hydroxides and clays that fill and occlude pores in sandstones. Na released from the feldspar albite appears to have been absorbed by the algal biofilm, or its release was inhibited by the algae. These observations suggest that algae have no direct influence upon feldspar weathering. However, the maintenance of a wet biofilm by algae may enhance the water-holding capacity of masonry surfaces, encouraging feldspar dissolution by inorganic reactions.

Experimental charges lacking Mg-bearing minerals tended to show release of Mg when algae were present. This is thought to result from death and decay of the algae and consequent release of Mg-porphyrin rings from chlorophyll. High, unsupported aqueous Mg concentrations may, then, be a useful measure of the health of the algal culture in experiments like he ones undertaken here.

As algae is an autophototroph it is often one of the first organisms to colonise natural building stone and many of its exudates of the would be utilised as a nutrient source by higher organisms, such as fungi or lichen, which are known to have a much higher degradation potential than the algae.

## 5 CONCLUSIONS

1. We have developed an experimental protocol that enables detection of algal influences upon the dissolution of cations from the surfaces of minerals commonly found in natural stone masonry. Use of powdered samples to enhance solute yields combined with high precision ICP-AES chemical analysis allowed us to resolve small changes in concentration in the experimental aqueous medium. Pure (axenic) algal cultures and rigorous aseptic technique permitted examination of algae alone, without interference from known, more aggressive microbes such as bacteria and fungi. The results presented here are the first clear evidence that green algae alone may have a significant influence upon mineral dissolution.

2. Yields of aqueous cations were much larger for carbonates than silicates, both in the presence and absence of algae.

3. Algae tend to create a mildly alkaline aqueous environment. This may influence the dissolution rates of some mineral types such as calcite, where more alkaline conditions reduce dissolution rates.

4. Iron dissolution from carbonates is enhanced by algae. This may influence iron mobility during sandstone weathering and aid in the formation of iron-rich patinae.

5. Ca release from calcite was enhanced in the presence of algae, especially at lower pH. Carbonates such as calcite and siderite are important cementing microfabric elements in sandstones and limestones, and our results suggest that algal activity will enhance degradation of the masonry surface.

6. Algae appear to have no direct influence on feldspar dissolution, but maintenance of a humid environment by a biofilm at the masonry surface will, indirectly, lead to destruction of feldspars and possibly precipitation of pore-blocking clays or Al-hydroxides.

## 6 ACKNOWLEDGEMENTS

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# The Mechanism Of Photo-Oxidative Degradation Of Acrylonitrile-Butadiene-Styrene (Abs) Resins Used In Pipes

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**Summary: This paper investigates the mechanism of ultraviolet (UV) degradation in ABS resins. ABS specimens were subjected to accelerated natural weathering at an ambient temperature to separate thermal effects from the mechanism of UV degradation. Degradation primarily occurs in the polybutadiene (PB) phase of ABS due to the polymer microstructure. Fourier transform infra-red (FTIR) shows that degradation of the PB phase is predominantly located at the surface of the polymer and that bulk degradation does not occur. Changes in chemical structure observed by FTIR are associated with oxidation and the formation of photoproducts, corresponding to a carbonyl peak at 1721 $cm^{-1}$ and hydroxyl peak at ~ 3465 $cm^{-1}$. The absorbance bands at 966.92 $cm^{-1}$ and 911.43 $cm^{-1}$ corresponding to the trans C=C unsaturation (vinyl) in polybutadiene, and the 1,2-butadiene terminal vinyl C-H band respectively, were distorted indicating bond un-saturation. The oxidation induction time (as determined by chemiluminescence) was significantly lower at the surface of weathered ABS compared to the bulk polymer. Dynamic Mechanical Thermal Analysis (DMTA) showed an increase in the glass transition temperature for UV-exposed specimens, reflecting an increase in Young's modulus due to crosslinking in the PB phase. Under plane strain loading conditions, brittle fracture occurs when surface degradation reaches a critical depth. To develop a technique for lifetime prediction after weathering, fracture mechanics theory was used to relate UV exposure time to an equivalent surface notch depth. Assuming that this relationship is geometry-independent, it was used to estimate the reduction in critical hoop stress for ABS pipelines under hydrostatic pressure**

**Keywords: ABS, weathering, degradation, fracture mechanics**

## 1 INTRODUCTION

Acrylonitrile Butadiene Styrene (ABS) pipes are often used above ground in exposed applications. Although heat stabilisers are commonly included in ABS to prevent thermal degradation during processing and subsequent use in high temperature environments, stabilisation against weathering and the effect of ultraviolet (UV) radiation is often overlooked. Exposure to UV radiation induces changes in the polymer microstructure, which lead to polymer oxidation and eventual degradation.

Whilst photo-oxidative degradation of ABS resin is reported to primarily occur in the polybutadiene (PB) phase (Gesner 1965, Heaps 1968, Salman and Al-Shama'a 1991, Jouan and Gardette 1992), degradation can also occur in the Styrene Acrylonitrile (SAN) phase (Davis and Sims 1983, Jouan and Gardette 1992). According to Jouan and Gardette, degradation of the PB phase induces degradation of the SAN phase (Jouan and Gardette 1992). Photo-product species formed by photo-oxidation of the PB phase are strongly dependent on irradiation wavelength (Jouan and Gardette 1992). Photo-oxidative degradation of the polystyrene component of the styrene-acrylonitrile (SAN) copolymer also yields chromophores (Davis and Sims 1983, Jouan and Gardette 1992). Kulich and Gaggar conclude that initial colour fading is due to oxdation in the SAN phase, and subsequent yellowing discolouration is due to oxidation of the PB phase (Kulich and Gaggar, 1996). The level of SAN copolymer in an ABS blend is known to affect mechanical properties by altering the rate of oxygen permeation. Increasing the level of SAN copolymer decreases oxygen permeability and retards degradation of the dispsered PB phase.

Photo-oxidation of the PB phase is considered to be the major cause of mechanical property deterioration in ABS. For ABS exposed to ultraviolet radiation, photolysis of the methylene bond in the trans-1, 4-polybutadiene structure occurs, yielding an allylic radical polymer chain. The radical may undergo several reaction paths to yield further radicals together with oxygenated species such as hydroperoxides, ketones and esters (Shimada and Kabuki 1968, Shimada et al 1972). The rate of photo-oxidation is proportional to the intensity of irradiation and the log of the irradiation wavenumber (Shimada and Kabuki 1968).

With further UV exposure, stable oxygenated photoproducts decompose to yield oxygenated radical species. Photo-oxidation is accelerated by the presence of residual hydroperoxides formed during thermal processing, which may also decompose to produce other degradation products and radical chains. Crosslinking of the PB phase after radical formation is thought to contribute to the embrittlement of the PB phase (Scott and Tahan 1977). Several approaches exist to prevent photo-oxidation, including the use of UV stabiliser additives (Kulich and Gaggar 1996, Zahn 1997 Shimada and Kabuki 1972). Additives absorb the energy of irradiation and transfer it to the ABS resin by lattice vibration excitation. However since they are not able to completely absorb UV energy, bond disassociation of the methylene-methylene structure remains possible. The inclusion of pigments or additives intended for UV stabilisation may also reduce the impact resistance of the resin (Kelleher *et al.* 1967, Davis and Gordon, 1974).

The relatively high impact strength of some ABS resins can be attributed to an increase in the PB phase content, which is susceptible to photo-oxidation (Heaps, 1968, Kelleher *et al.* 1967). Zahn shows that the impact strength of high impact grade UV-stabilised ABS, decreases to ~ 70% of its original impact strength after UV exposure (Zahn 1997). In conditions of extreme UV exposure, it is recommended that the surface of ABS structures be painted to prevent photo-oxidative degradation (Shimada and Kabuki 1972, Gugumus 1979, Kurumada *et al* 1987). For thick ABS structures, degradation due to weathering is confined to the surface due to the formation of an initial degradation layer which prevents oxygen diffusion and further UV penetration. Subsequent oxidation of the bulk polymer is prevented and an embrittled surface layer forms, which eventually causes bulk failure under applied loading (Kulich and Gaggar 1996).

The critical performance requirement of thick-walled ABS pipe for outdoor water transport, is to withstand the internal hydrostatic pressure to which it is rated. A degraded surface layer on the pipe will contain numerous micro-cracks which may be sufficiently large to propagate into the pipe wall under an applied load (Kulich and Gaggar 1996, Tiganis *et al* 2001). In this study, thick ABS specimens (which was not UV stabilised), were weathered under accelerated conditions closely matching those of natural weathering. Changes in chemical and mechanical properties were assessed, and the influence of weathering on failure was analysed using fracture mechanics theory. An empirical relationship between UV-exposure time and an 'equivalent' surface notch depth is derived and used to estimate reduction in critical hoop stress for weathered ABS pipelines.

## 2 EXPERIMENTAL PROGRAM

### 2.1 Weathering

Specimens from a commercial ABS pipe resin comprising a SAN-graft-PB bimodal matrix with dispersed ungrafted PB polymer, in the form of impact (AS, 1146.1) and tensile (AS, 1145) specimens were weathered in an Atlas Ci 2000 weatherometer (Xenon Arc - 0.55 W/m$^2$/nm @ 340 nm) at ambient temperature (30ºC) such that the mechanism of degradation was not affected by heat (Tiganis *et al 2001*). Specimens were exposed using accelerated conditions for a 3-month duration with periodic removal for assessment. Table 1 shows the real accelerated weathering times and levels of UV exposure and the corresponding simulated times that are referred to in this paper. Simulated times were determined following Martin (Martin, 1977) and using an average of several years of total UV radiation data collected in Melbourne, Victoria and Allunga, North Queensland Australia, using an Eppley pyranometer. Based on the chosen accelerated conditions, a 3-month exposure period (1973 hours) approximates to a 1-year period of natural exposure.

**Table 1: Accelerated weathering times and UV exposure levels with the corresponding simulated natural weathering times that are referred to in the text**

| Real Instrument time (hours) | UV exposure (MJ/m$^2$) | Simulated natural weathering time |
|---|---|---|
| 41 | 7.5 | 1 week |
| 82 | 15 | 2 weeks |
| 164.5 | 33 | 1 month |
| 329 | 66 | 2 months |
| 493 | 100 | 3 months |
| 658 | 133 | 4 months |
| 986 | 200 | 6 months |
| 1480 | 300 | 9 months |
| 1973 | 400 | 12 months |

### 2.2 Microstructural property assessment

Control and weathered ABS specimens were visually inspected using an Olympus optical microscope. To assess the effects of weathering on polymer microstructure, 20µm thick specimens of ABS were microtomed from both the surface and bulk of sample that had been weathered for 12 months. These specimens were assessed by Fourier Transform Infra-Red (FTIR)

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spectroscopy to identify the carbonyl and hydroxyl absorptions formed due to the oxidation of weathered ABS and also to identify the characteristic infra-red absorptions of vinyl groups in control ABS specimens. The specimens were also analysed using an Atlas CL 400 chemiluminescence instrument to investigate the chemical stability of the surface of the weathered ABS. The samples were analysed at an isothermal temperature of 180°C in oxygen, after a nitrogen pre-phase. The onset of polymer degradation, or the oxidation induction time (OIT) was recorded for each sample.

### 2.3 Mechanical property assessment

The glass transition ($T_g$) temperature and loss moduli of the PB phase for ABS specimens before and after weathering were measured using a Rheometrics Scientific Solids Analyser RSA II DMTA. Instrumented impact analyses were performed on control and UV exposed ABS specimens, using a Radmana impact tester in accordance with AS 1146.1 (AS, 1146.1). To compare the effects of notching and UV exposure on the failure of ABS under static bending stresses, a hydraulically-driven Materials Testing System (MTS) 810 in a three point bending mode was also used. Notched (0.5 mm) control specimens were prepared according to AS 1146.1 and subjected to various nominal bending stresses. Exposed specimens (6, 9, 12 months) were also subjected to a static bending stress until failure and failure times were compared. Specimens were oriented such that tensile bending stresses acted on the UV-exposed or notched surfaces.

## 3 RESULTS & DISCUSSION

### 3.1 The effect of weathering on the chemical properties of ABS

#### 3.1.1 Optical microscopy and visual examination

A yellow-brown discolouration was observed to develop after 2 weeks of simulated natural weathering time under accelerated conditions, for the ABS polymer. Discolouration was localised at the surface and was of very minimal depth, with fading and slight yellowing occurring initially. The depth of discolouration increased with exposure time, reaching approximately 0.1 mm after 12 months of simulated natural weathering, at which point yellowing was prominent. Discolouration is often related to the coupling of radical scavengers with degradation peroxy radicals (Faucitano *et al.* 1996). However for ABS, discolouration is attributed to the formation of photoproduct chromophores during degradation. Chromophores absorb energy in the UV-visible spectrum and cause discolouration of the resin (Kulich and Gaggar 1996, Salman and Al-Shama'a 1991). According to the literature, both the PB and SAN phase are said to form chromophores upon oxidation which are responsible for fading and yellowing.

#### 3.1.2 Chemical stability - Oxidation Induction Time

Degradation at the surface of ABS causes an increase in polymer density, limiting oxygen diffusion and thus decreasing the rate of degradation through a thick ABS specimen, such that degradation is absent in the bulk polymer (Lemaire *et al*, 1996). If UV stabiliser is present in commercial grades of ABS, it must first be depleted at the surface for degradation to occur (Faucitano *et al* 1996, Heaps 1968, Clough *et al*, 1996). Figure 1 shows chemiluminescence traces for control polymer, and polymer at the surface and bulk of ABS weathered for 12 months. The voltage trace recorded in chemiluminescence assessment is directly related to light emitted by the decay of hydroperoxides, formed in the polymer due to photo-oxidation, upon heating. (Clough *et al*, 1996).

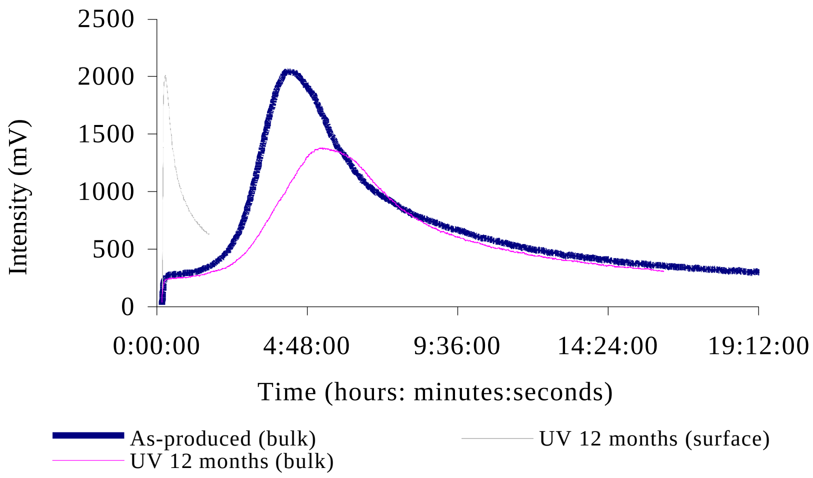


**Figure 1: Chemiluminescence traces for control ABS polymer and polymer from the surface and the bulk regions of weathered ABS**

As shown, the oxidation induction time (OIT) is almost instantaneous for the surface polymer of the weathered ABS specimen. This was expected as the resin was not UV stablised and degradation occurred upon exposure. In contrast, the signal for the bulk polymer of the same specimen (OIT ~ 170 minutes) is similar to that obtained for control specimens (153 minutes), indicating that no degradation occured in the bulk polymer.

3.1.3 FTIR analysis of polymer microstructure

FTIR spectroscopic analysis was used to study weathering-induced chemical changes in the microstructure of ABS. Results from control polymer specimens were compared to the surface and bulk polymer of ABS weathered for 12 months (simulated natural weathering time).

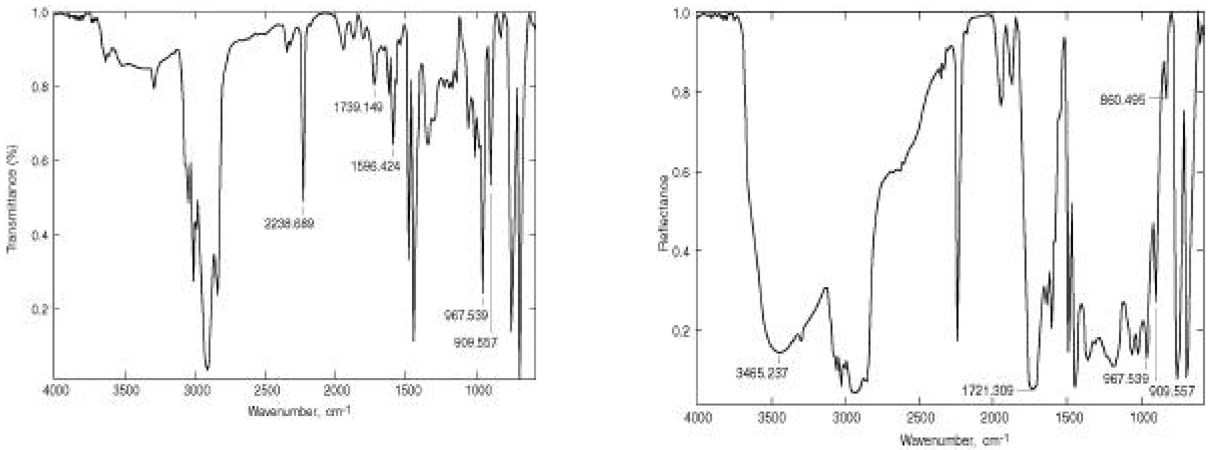


**Figure 2. FTIR spectra of the bulk (left) and surface (right) polymer of a ABS weathered for 12 months**

As shown in Figure 2, the spectrum representing the bulk analysis from the exposed polymer replicates a trace that is typical for control ABS, with no observed changes in the characteristic infrared absorptions and no evidence of degradation products. In contrast however, spectral changes are clearly observed at the surface of the aged polymer, in particular the carbonyl peak at 1721 $cm^{-1}$ and hydroxyl peak at ~ 3465 $cm^{-1}$ indicating changes in chemical structure associated with oxidation and the formation of photoproducts. The absorbance bands at 966.92 $cm^{-1}$ and 911.43 $cm^{-}$1 correspond to the trans C=C unsaturation (vinyl) in polybutadiene, and the 1,2 butadiene terminal vinyl C-H band respectively. The surface spectrum for weathered ABS shows a significant distortion in these bands, indicating changes in the PB microstructure, which are attributed to chain scission and cross-linking (Shimada and Kabuki 1968).

## 3.2 The effect of weathering on the mechanical properties of ABS

3.2.1 Effects of weathering on the glass transition of ABS

The effects of weathering on the microstructure and the thermo-mechanical response of the PB phase in ABS were investigated using dynamic rheology. For weathered specimens, increases in both the loss modulus (E") and tan $\delta$ parameters were observed, indicating an increase in the glass transition temperature ($T_g$) of the PB phase. These increases reflect an increase in modulus due to cross-linking of the PB phase (Kulich and Gaggar 1996). This will be confirmed in the future with further studies based on molecular weight distribution.

3.2.2 Effects of weathering on impact properties of ABS

Figure 3 shows the detrimental effect of weathering on the impact strength of ABS resin. The initial impact resistance of 214 $kJ/m^2$ decreases to approximately 60 $kJ/m^2$ within 1440 hours (2 months) of simulated natural weathering. Failure of exposed ABS is dependent on the extent of surface degradation and at a critical depth of degradation an abrupt failure occurs, as seen in Figure . Kulich and Gaggar (Kulich and Gaggar 1996) also observed a similar trend in the reduction of impact strength. Similar to thermo-oxidative degradation of ABS, photo-oxidation of ABS causes a sudden loss in impact strength that is critically dependent on the depth of surface degradation (Tiganis *et al* 2001). Zahn reports similar findings for ABS that is not UV stabilised, where impact strength falls to approximately 30% of the original value after 1500 hours of UV exposure (Zahn 1997).

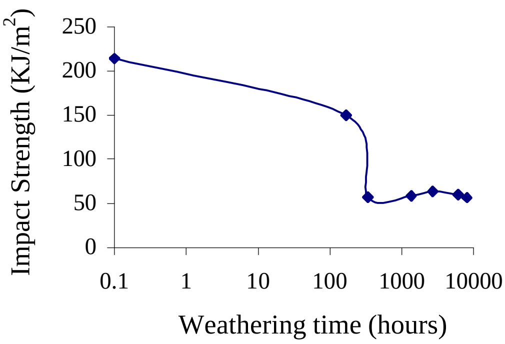


**Figure 3: The effects of weathering on the impact strength of ABS resin**

### 3.3 The effects of weathering on the failure mechanism of ABS

Results from the previous section suggest that weathering ABS creates a degraded surface layer that precipitates bulk mechanical failure similar to the action of a sharp surface notch. To investigate this hypothesis, the failure mechanism of UV-exposed specimens subjected to static bending stresses was examined. Figure shows the deflection/time curve for UV-exposed and virgin ABS specimens subjected to a static bending stress of 65 MPa.

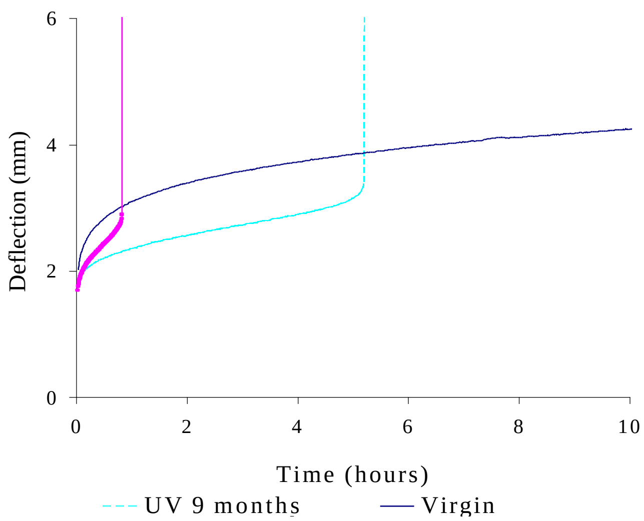


**Figure 4: The variation of crosshead deflection with time for static load SENB tests on UV-exposed and un-notched virgin ABS specimens**

As shown, the curve for un-notched virgin ABS is characteristic of creep deformation, an initial sharp increase in deflection, followed by relatively slow deformation. Failure eventually occurred by plastic collapse after three months. In contrast, whilst the initial sharp increase in deflection is observed in the UV-exposed specimens, the magnitude of deflection is reduced. This can be attributed to the increased stiffness of the degraded layer reported previously (Tiganis *et al* 2001). Furthermore, a transition is observed, after which deflection increases sharply. This typically indicates fracture failure with an increase in specimen compliance after crack initiation and subsequent propagation through the specimen thickness. Visual examination of failed specimens also suggested that brittle fracture occurred. Crack initiation was observed at UV-exposed surfaces, and separated surfaces appeared smooth with only limited stress whitening. Such features are typical of brittle failure from surface defects, with restricted plastic deformation in the bulk polymer.

Assuming that only limited plastic deformation occurs in the bulk polymer, Linear Elastic Fracture Mechanics (LEFM) theory can be used to relate UV exposure time to an 'equivalent' surface notch depth. For a single-edge-notched bend (SENB) specimen, an applied stress intensity factor (SIF) can be written as (Hashemi and Williams 1984)

$$K_I = Y\left(\frac{6M}{BW^2}\right)a^{1/2} \qquad (2)$$

where M is the applied bending moment (Nm). B is the specimen thickness (m); W is the specimen width (m) and a is the surface notch depth (m). Y is a geometric correction factor which accounts for the influence specimen size and loading type over $K_I$. At some stage during a test, a crack will initiate from the surface notch and slowly grow through the specimen thickness towards the free surface. Under plane strain conditions, the criterion for eventual brittle fracture can be written as

$$K_I = Y\left(\frac{6M}{BW^2}\right)a_c^{1/2} = K_{IC} \tag{3}$$

where $a_c$ is the critical crack length, and $K_{IC}$ is the material plane strain fracture toughness. By applying different static bending moments to SENB specimens, a range of failure times is produced and eq. (3) can be used to determine the variation of $K_{IC}$ with time (Figure )

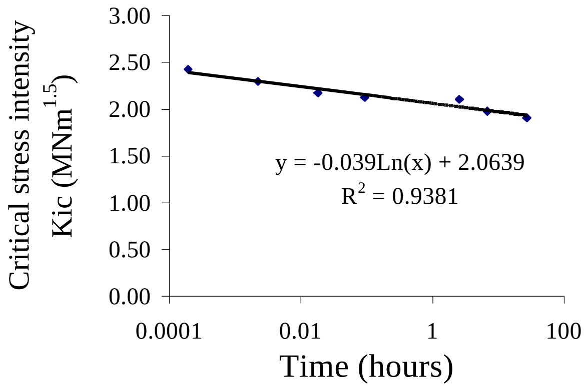


**Figure 5: Critical stress intensity factor ($K_{IC}$) for un-exposed SENB specimens**

As shown, the time-dependence of $K_{IC}$ for *un-exposed* ABS specimens can be approximated as

$$K_{IC} = -0.04\ln(t) + 2.06 \tag{4}$$

where $K_{IC}$ is in MPa m $^{0.5}$ and $t$ is in hours. If we assume that failure in UV-exposed specimens also occurs when $K_I = K_{IC}$, failure times for UV-exposed specimens can be used in equations (4) and (3) to obtain a relationship between UV exposure time and 'equivalent' notch depth (Figure ).

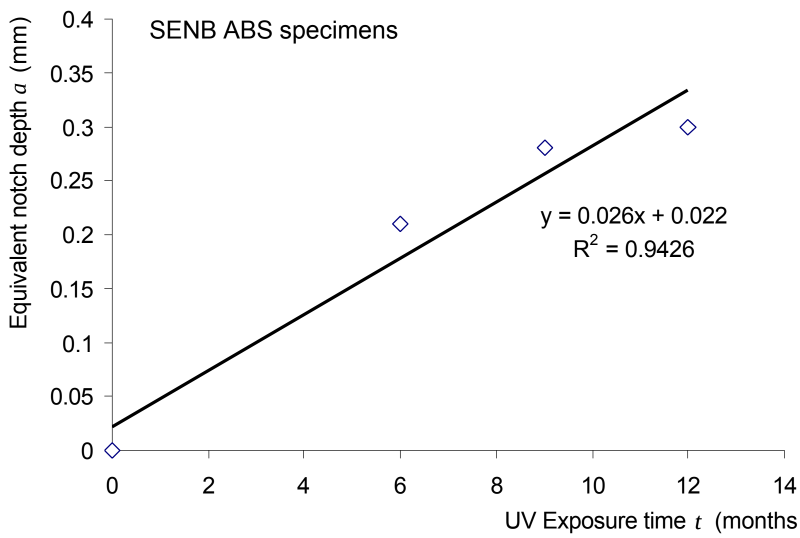


**Figure 6: Equivalent notch depths determined from failure times in UV-exposed SENB specimens**

As shown in figure 6, the resulting relationship from three-point bend tests is given by

$$a = 0.03t - 0.02 \tag{5}$$

where equivalent notch depth $a$ is in mm and exposure time $t$ is in months.

### 3.4 The effect of weathering on ABS pipe under hydrostatic pressure

Having derived an empirical relationship between τ and a, it can be applied to pressure pipelines if we assume that eq. (5) is independent of specimen geometry. If this is true, then subjecting a pipe to the same UV exposure as a rectangular bar specimen will result in the same equivalent external surface notch depth. Whilst current work investigates the validity of this theory, we now go on to suggest how UV exposure may influence the performance of ABS pipes under hydrostatic pressure.

Let us assume that a pipe has been weathered in service for a known period, τ, and we require the reduction in its hydrostatic pressure capacity. If an equivalent external surface notch depth can be determined from eq. (5), the corresponding applied SIF ($K_I$) associated with a pipe under internal pressure is given by (Rooke and Cartwright 1974)

$$K_I = y\left(\frac{2pR_i^2\sqrt{\pi a}}{R_i^2 - R_o^2}\right) \qquad \textit{(6)}$$

y is a geometric correction factor, p is the internal pressure (Pa), $R_i$ is the pipe inner radius (m), $R_o$ is the pipe outer radius (m) and a is the equivalent notch depth determined from eq. (5). As before, failure will occur when $K_I = K_{IC}$ as given in eq. (4). Figure shows the predicted effects of weathering on ABS pipe. Equivalent notch depths between 1.6-1.9 mm, correspond to weathering periods of 4.5-5.3 years simulated natural exposure.

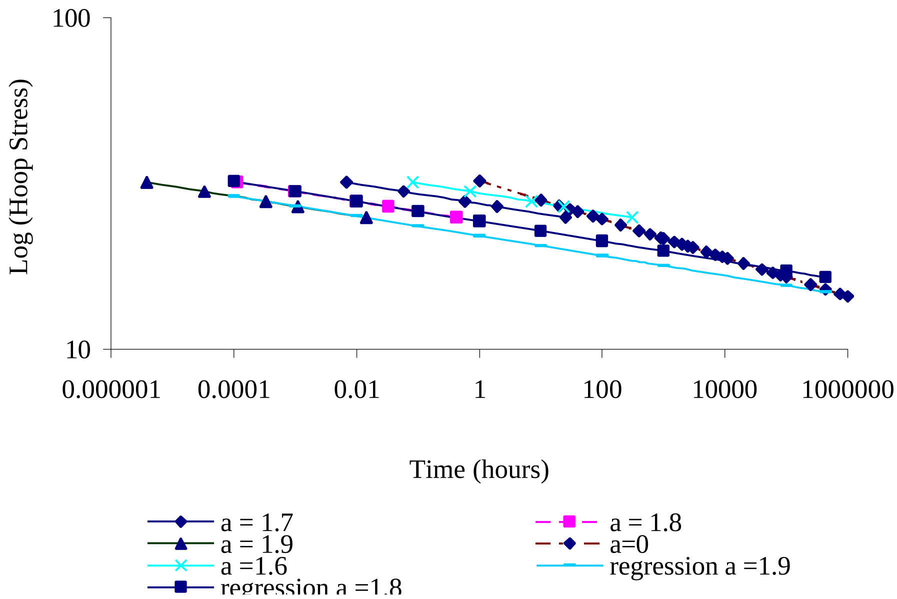


**Figure 7 : The predicted effect of weathering (and the equivalent surface notch) on the long term hydrostatic performance of ABS**

As shown, increasing UV exposure is predicted to reduce the time to failure for a given applied hoop stress. Whilst these predictions should be validated by experimental investigation, they may highlight the importance of UV-stabilisers in ABS pipes used in outdoor applications.

However, it should be noted that the results in figure 8 do not indicate *in-service* lifetimes. They indicate the performance of pipes that are *no longer* subjected to UV degradation *after* a period of exposure. In-service lifetime prediction can be idealised as a 'competition' between UV degradation at the pipe outer surface (which in turn produces an equivalent notch depth) and slow crack growth under the applied hydrostatic stress. During slow, stable crack growth, the applied SIF can be related to crack growth rate (*da/dt*) by a 'power law' relationship (Sandilands and Bowman 1986)

$$\frac{da}{dt} = AK_I^{\,m} \qquad (7)$$

Where A and m are constants for a particular environment and are dependent on material visco-elasticity. Figure schematically illustrates the transition between UV-controlled degradation and slow crack growth for a pipe in service.

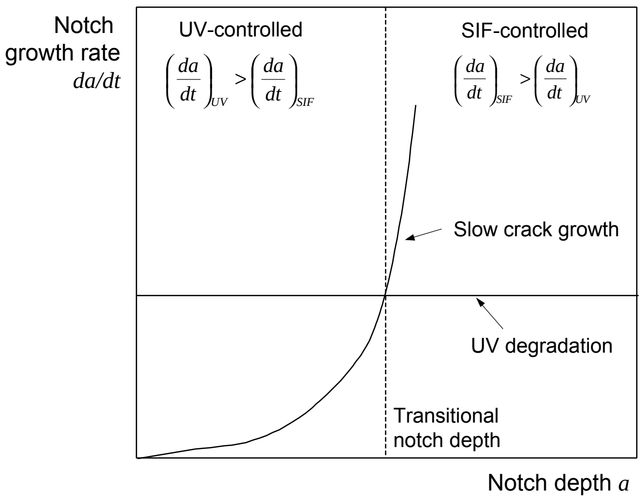


**Figure 8: Schematic diagram of the transition between UV degradation and slow crack growth in service**

As shown, the constant equivalent notch growth rate attributed to UV degradation is obtained by differentiating the linear relationship in eq. (5), and the ‘mechanical’ slow crack growth rate is described by eq. (7). Whilst notch growth is initially controlled by UV degradation, it becomes SIF-controlled above a transitional notch depth, which must be determined for in-service lifetime prediction. Although the initial results in this study suggest that equivalent notch growth under UV degradation proceeds at a constant rate, the effect of limited oxygen diffusion must be considered. For example, after extensive periods of UV exposure, the depth of the degradation in ABS will cease to grow with exposure time. Consequently, the empirical relationship between equivalent notch depth and exposure time may be misleading. Current work at CSIRO is progressing towards separate relationships between degradation depth/equivalent notch depth, and degradation depth/UV exposure time. It is anticipated that the growth of an equivalent notch can then be related more accurately to the kinetics of the UV degradation process.

## 4 CONCLUSIONS

Photo-oxidative degradation of the PB phase in ABS occurs in the absence or depletion of residual UV stabiliser, and ultimately precipitates mechanical failure of the polymer. Degradation of the elastomeric PB phase in ABS is initiated via photolysis of the methylene bond in the trans-1, 4-polybutadiene polymer structures, producing radicals that oxidise to produce carbonyl and hydroxyl products. Cross-linking of polymer chains, is also facilitated by free radicals. Degradation processes such as crosslinking cause an increase in polymer density at the polymer surface, preventing further penetration of UV light and oxygen into the bulk polymer. Thus degradation in thick ABS structures is localised at the surface. The mechanism of ABS photo-oxidative degradation in the PB phase is based on auto-oxidation and is strongly dependent on the wavelength and intensity of irradiation. The kinetics of degradation are not expected to be typically Arrhenius due to the limitation of oxygen diffusion.

In weathered ABS, the contribution from the dispersed PB phase to the overall ductility of the polymer, by particle cavitation, or localised shear yielding/crazing in the SAN matrix between PB particles, is greatly reduced due to degradation of that phase. Under impact loading, micro-cracks initiate from existing flaws in the degraded polymer surface layer. When the degraded layer reaches a critical depth, these cracks are sufficiently large to propagate into the bulk of the polymer causing abrupt mechanical failure.

Under static loading, the effect of UV degradation was compared to that of a sharp surface notch. Deflection/time curves and failure surface examination of degraded specimens indicated crack initiation and subsequent brittle propagation from the UV-exposed surface. Using plane strain fracture toughness measurements on un-exposed SENB specimens, an empirical relationship between UV-exposure time and an equivalent surface notch depth was determined. In the absence of experimental validation, it was assumed that this relationship was geometry independent and could be applied to pipelines under hydrostatic pressure. The variation of critical hoop stress with time was generated, indicating how a period of UV exposure may reduce failure times after subsequent UV protection. The in-service lifetimes of UV-exposed pipes was idealised as a competition between the growth rate of an equivalent notch under UV degradation, and the slow crack growth rate under an applied SIF. It was proposed that UV-controlled notch growth becomes SIF-controlled above a transitional notch depth.

Whilst the current study may suggest that weathering stabilisers or protective paints are required for ABS pipes that are under UV exposure, further investigation is required. The effect of degradation kinetics on equivalent notch growth must be considered, and the relationship between exposure time and notch depth must be validated for different test geometries.

## 5 ACKNOWLEDGEMENTS

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# Durability Performance Of Polymer Concrete In Strongwall Construction

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**Summary: The concept of a polymer concrete based innovative construction system has been developed by Strongwall. This system enables speedy and economical construction of internal and external load bearing as well as non-load bearing walls. Research is focussed on identifying various materials that could be used for manufacturing the Strongwall system. Two polymer concrete formulations have been developed using industrial wastes.**

**Before the system is manufactured commercially, it is essential that its material and engineering characteristics are assessed and documented to aid design. This is even more compelling when the new material is manufactured using industrial wastes. Accelerated weathering tests were conducted using highly concentrated sodium chloride, sodium sulphate, sulphuric acid and sodium hydroxides. Circular disc specimens were prepared from cast cylinders and splitting tensile strength was evaluated after accelerated weathering to assess the degradation of strength.**

**The results indicate that the tested polymer concrete formulations have significant resistance to environmental degradation. It is noted that high concentration of sulphuric acid and sodium hydroxide can be harmful.**

**Keywords: Polymer Concrete, Durability, Strong Wall, Splitting Test, Environmental Degradation**

## 1 INTRODUCTION

Conventional brick masonry walls are a popular and economical way to enclose modern buildings. However, over the past decade an increasing number of masonry veneer problems have come to plague building designers and owners around the world. To overcome some of the problems associated with traditional construction, new concepts of construction methods and materials have been explored. The innovative construction system developed by Strongwall combines the advantages of both the construction method and a new lightweight material to enable speedy and economical construction.

The new system could be used for both internal and external load bearing as well as non-load bearing walls. Besides its adaptability and economy, the construction system has been designed for durable residential houses in regional and remote outback areas where construction resources are in scarcity, for use in cyclone regions where strength and durability are essential, for external cladding in multi-storey residential & commercial buildings etc.

Polymer concrete has been selected on the basis that its relative strength is higher than conventional concrete, and it has the ability to cure at a faster rate. Further, if appropriate materials are chosen then a high strength to weight ratio could be achieved. In recent times, construction materials with ecological characteristics have generated great interest. From this point of view, polymer concrete is very attractive as it can be manufactured using recycled and/or industrial waste materials.

Three types of concrete polymer composites are popular throughout the world primarily due to their high strengths and durability. The materials are:

- Polymer impregnated concrete (PIC) which consists of a precast Portland cement concrete impregnated with a monomer system that is subsequently polymerised in-situ.
- Polymer cement concrete (PCC) with polymeric admixtures to green concrete in the form of latexes or plasticisers.
- Polymer concrete (PC) which consists of an aggregate mixed with a monomer or resin that is subsequently polymerised in place.

All these have strength and durability properties that are considerably better than that of Portland cement concrete. Also, the behaviour of concrete polymer composites is substantially different to that of normal cement concrete since the former uses a viscoelastic resin binder while the later uses an organic cement binder.

The characteristics of polymer composites depend upon various factors such as the type and amount of binder, the amount, size and type of aggregates and fillers etc. The ability to fill the voids between solid particles by fine fillers and to disperse in the mixture is considered to be vital to achieve high strength and stiffness.

At present, the Division of Building, Construction & Engineering of CSIRO is engaged in a sponsored research project. The primary aims of the research are to identify and develop a suitable material (or an array of materials) for the construction of the system and assess its (their) characteristics. While developing the materials reported in this paper, a large number of exploratory tests were conducted using various types of industrial wastes. However, due to lack of space, only characteristic properties of two types of polymer concrete are reported in this paper. Also, the durability performances under simulated aggressive forces of nature are included.

## 2 EXPERIMENTAL STRATEGY

As mentioned above only two types of materials are reported in this paper. Both materials are designed to achieve 40 MPa compressive strength. The mechanical properties of the two materials are shown in Table 1. The ingredients used are not disclosed here due to the commercial nature of the product.

Four test cylinders (76 mm diameter x 150 mm high) were cast from each mix type. They were initially cured at 35ºC for 45 minutes and then at 25ºC for 5 days. Results obtained from the initial tests indicated that the specimens gained full strength within 3 to 7 days. The disc specimens were primarily grouped into two categories, namely control specimens and specimens to be subjected to accelerated weathering. The accelerated weathering was simulated by subjecting the specimens into wetting and drying cycles in an aggressive environment. The specimens subjected to the accelerated weathering conditions were also divided into two groups. The specimens in one of the groups were only subjected to fan drying under the laboratory conditions, whereas the specimens in other group were oven dried after every five cycles for weight measurements. The control specimens and the specimens which were not subjected to oven drying were finally tested for strength tests.

**Table 1. Material Types Investigated**

| *Material Type* | *Type A* | *Type B* |
|---|---|---|
| Compressive Strength (MPa) | 40MPa | 40MPa |
| Flexural Tensile Strength (MPa) | 10.5MPa | 10.7MPa |
| Density (kg/m$^3$) | 1440kg/m$^3$ | 1530kg/m$^3$ |

## 3 EXPERIMENTAL PROCEDURE

Approximately 10 mm thick circular specimens were cut and placed back together in their original positions within the cylinder. Each test specimen (circular disc) was numbered sequentially in the cylinder for future reference. Altogether 32 circular discs were prepared for the investigation.

For the durability tests, some of the specimens (circular discs) were subjected to accelerated weathering while others were left as control specimens.

### 3.1 Durability Assessment

In order to understand the durability characteristics, an appropriate test as well as a performance indicator is needed. The type and nature of the test will depend upon its intended use and a wide range of variables must be considered in developing the test. Further, the accelerated laboratory experiments should be corroborated using long term monitoring of the in-situ durability performance of the material.

As a consequence of the brittle nature of polymer concrete, flexural tensile strength has been widely used as a performance indicator. In this study splitting tensile strength of the material under diametral loading was selected as the performance indicator due to the following reasons.

- Simplicity of the test set up
- Several test specimens (1 cm thick discs) can be prepared from the same cylindrical specimen
- Effect of the variability of the material in different test specimens can be minimised since several test specimens can be prepared from the test cylinder.
- Because specimens are relatively small, it is easy to perform the drying and wetting cycling tests.
- Cylindrical specimens are large enough to be representative of the material as a whole
- Effect of the aggressive agent can be assumed uniform through the thickness of the test specimen because of its thinness.
- To the authors knowledge, this is the first time the splitting test method has been used for the assessment of strength degradation of specimens subjected to exposure. Therefore, this study would also allow assessing the suitability of the test technique to such applications.

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Continuous assessment of deterioration is a difficult task, as the deterioration is progressive and small in its early stages. Visual inspection, change in volume/dimensions and change in weight are some of the commonly used parameters to assess the degree of deterioration.

The weight loss or gain of the specimens was monitored in this study. Because of its non-destructive nature, weight loss/gain can be used as an indicator to identify the time at which the destructive splitting tests should be carried out.

### 3.2 Weathering Test & Agents

Most durability tests involve a prolonged period of soaking the specimens in a weathering agent, and then assessing the degree of deterioration. On using a reasonably strong weathering agent, the rate of deterioration will be slow, and therefore, the test itself must be of reasonably long duration. Also, it does not simulate what is happening in a real life situation, where both wetting and drying processes cause the degradation of the material.

An alternative approach to the above, is a shorter duration cyclic test where the specimen is immersed in the weathering solution for a specified time and then dried in a controlled environment. This process is repeated for a set number of cycles, or until measurable deterioration occurs. In this case, the rate of deterioration will be influenced by the concentration of the solution and severity of the cycling procedure. This approach reasonably simulates the real life performance.

For the current study, the cycling process as described above was selected and the following agents were used.

- Sulphuric Acid – 25%
- Sodium Hydroxide – 10 molar
- Sodium Chloride – 14%
- Sodium Sulphate – 14%

The Australian/New Zealand Standard AS/NZS 4456:1997 recommends the use of a mixture of 14% of Sodium Chloride and 6.2% of Sodium for the degradation assessment of masonry units due to salt attack. The same strength mixture was used for the current specimens to test for salt resistance ability. Under normal conditions, 25 % of Sulphuric Acid and 10 molar Sodium Hydroxide are highly unlikely to occur. However, to understand the worst case performance, these mixtures were used.

As the test progresses, the concentration of the sulphate/chloride irons and the pH value of the solution vary due to reactions with the sample. Unless these parameters are kept constant, they can influence the results and increase the length of the test.

It should be noted that whatever type of accelerated test method is used, a correlation must be obtained between laboratory and field performance so that the test method could be fine tuned for realistic conditions for future tests. Unfortunately, when materials are newly developed, there are no past records for comparison.

### 3.3 Test Procedure

Eight, approximately 10 mm thick, circular test specimens were prepared from each cylinder. A typical example is schematically shown in Fig. 1. The specimens 1 to 6 were meant for the splitting test and 7 & 8 for the weight loss assessment. The specimens used for the weight assessment were initially oven dried (at 105 ºC) and their datum weights were recorded. Then all the test specimens, other than the control specimens were subjected to 24-hour wet and dry cycling.

| *Spec. No.* | *Test Cylinder* | *Weight (gms)* | | *Legend* | |
|---|---|---|---|---|---|
| | | | | | |
| | | | | | |
| 1 | | 98.32 | | | Specimen to be Weathered |
| 2 | | 99.37 | | | and subjected to |
| 3 | | 99.88 | | | Splitting Test |
| 4 | | 99.58 | | | |
| 5 | | 102.91 | | | Specimen to be Weathered |
| 6 | | 102.79 | | | and subjected to |
| 7 | | 101.15 | | | Weight Measurement |
| 8 | | 100.88 | | | |
| | | | | | Control Specimen |
| | | | | | |

**Figure 1. A Typical Layout of Specimens prepared from a Cylinder**

The control specimens were left in the laboratory while others were subjected to 24-hour wet and dry cycling at the age of 7 days. For each cycle, they were immersed in the solution for about 2 hours and then fan dried for 22 hours in the temperature controlled (23 ºC) laboratory. Two replicates were tested for each test condition.

The specimens 7 & 8 were oven dried at every fifth cycle (except at the start when oven drying took place after 2 cycles) for weight assessment. At the end of every fifth cycle, fan drying commenced followed by oven drying for 2 hours. Then the specimens were cooled for about 5 minutes and weighed prior to the commencement of the next cycling period. The specimens intended for splitting tests were not oven dried.

After every 10 cycles the old solutions were replaced with fresh solutions. The wetting and drying cycles were terminated after 37 cycles, when some of the specimens indicated signs of deterioration by loosing weights.

## 4 RESULTS AND DISCUSSION

The weight loss/gain of specimens are shown in Fig. 2 and 3 (positive values indicate weight gain). Each result shown is the average of two replicates. The variability of the results produced by similar specimens was found to be insignificant, and therefore the results were averaged.

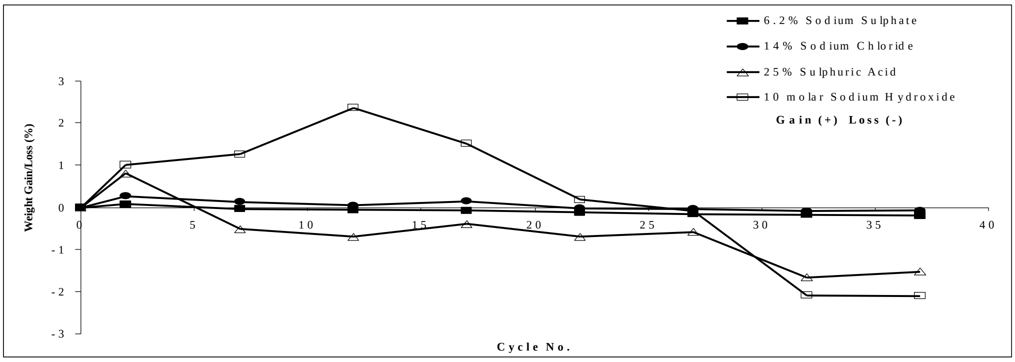


**Figure 2. The Effect of Accelerated Weathering on Material Type 1**

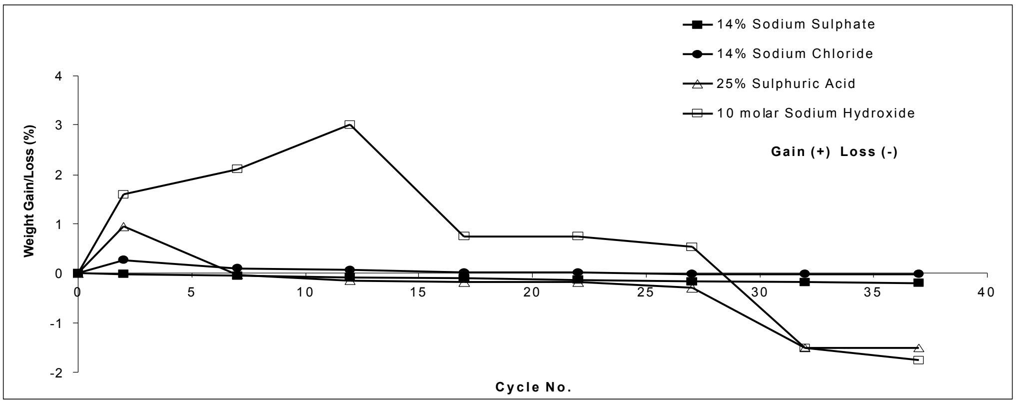


**Figure 3. The Effect of Accelerated Weathering on Material Type 2**

The following comments could be made on the above results:

- Behaviour of both material types is quite similar and is understandable as the materials are quite similar.
- Specimens do not show any significant deterioration with a severe exposure to sodium chloride and sodium sulphate solutions.
- The specimens subjected to severe sodium hydroxide exposure, initially gained weight, and subsequently, started to loose weight. The initial weight gain may be due to reaction products being deposited in the pores. As the specimen begins to deteriorate, a decrease in weight would have occurred due to spalling and similar effects. The signs of reaction taking place were observed by a noticeable change in colour. Initially, the specimens were grey in colour and after the first cycle, turned into a slightly yellowish colour.
- A significant deterioration of specimens, which were subjected to severe sulphuric acid attack, was noticed after the 27$^{th}$ cycle. Although there was no significant loss of weight initially, a considerable colour change has been observed, especially after oven drying at 105 ºC for 2 hours. The grey colour of the specimens was changed into a dark black colour. However, subsequent investigation has shown that the change in colour has occurred only on the surface. Fig. 4 shows the affected specimens under the influence of sulphuric acid together with two control specimens.

**Figure 4. Colour Change of the Specimens subjected to Sulphuric Acid**

The splitting test results are graphically shown in Figs. 5 and 6. The specimen numbers from 1 to 6 are consecutive specimens, and have been prepared from the same test cylinder. This enables comparison of the control specimens and the weathered specimens. The tensile splitting strength at the centre of the circular disc specimens was calculated using the equation shown below. The diameter and the thickness of the specimens were measured at three different locations and the average value was taken for calculations.

Splitting tensile Strength (MPa) = 2P/pDt
where,
P – Failure Load (N)
D – Diameter of the Specimen (mm)
t – Thickness of the specimen (mm)

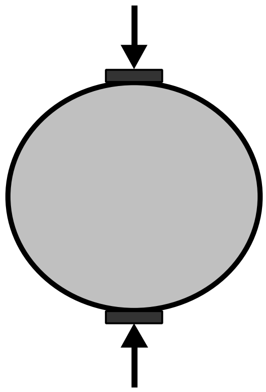

**Figure 5. Schematic Diagram of the Splitting Test**

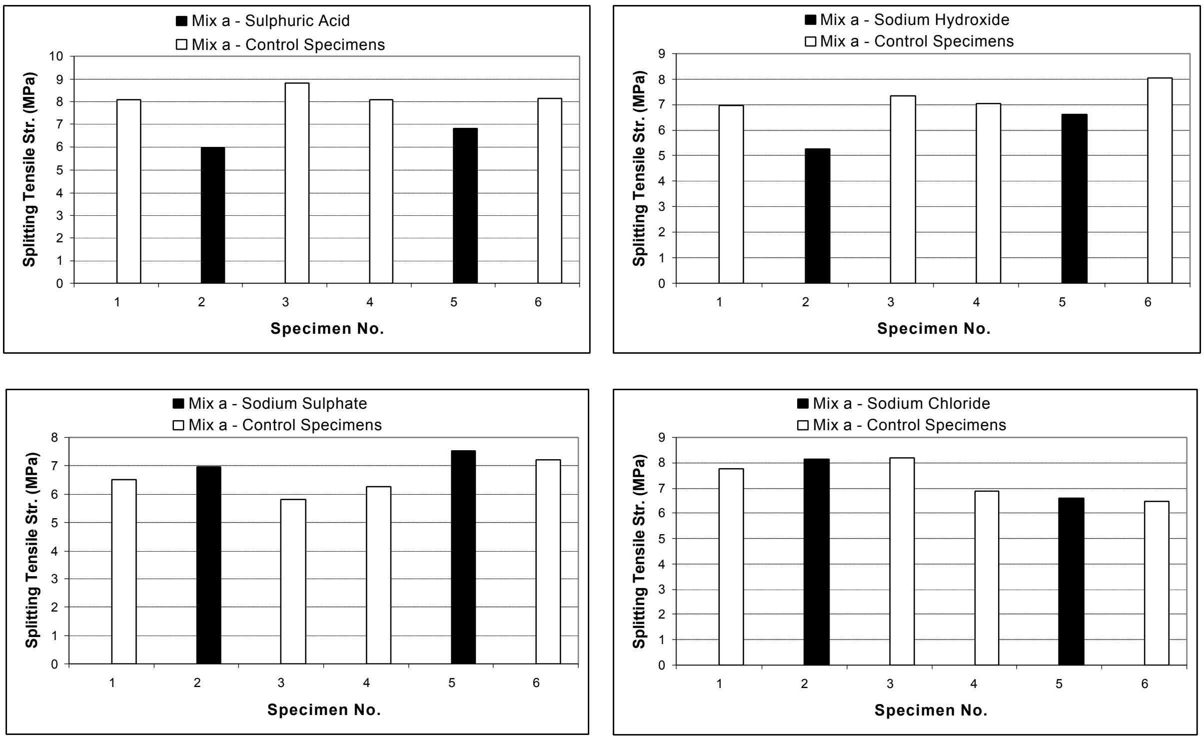


**Figure 6. Splitting Tensile Load of Material Type 1 Specimens subjected to Accelerated Weathering**

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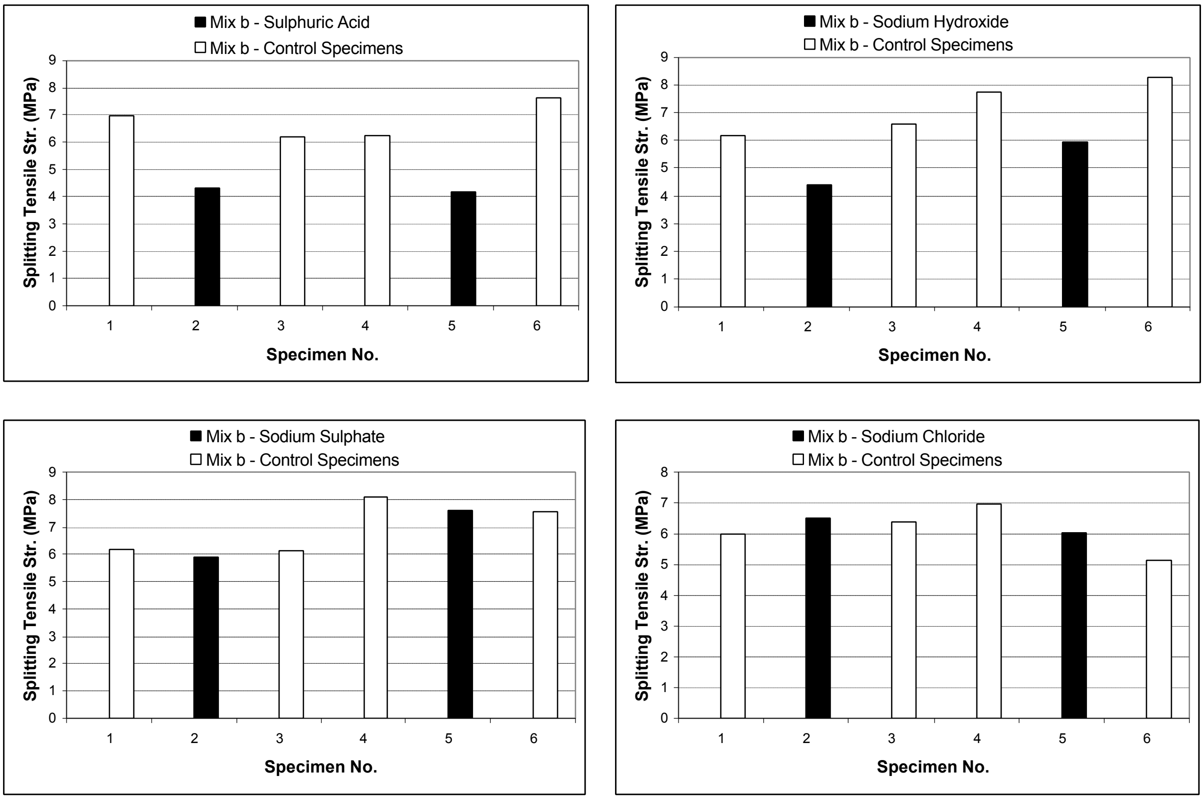


**Figure 7. Splitting Tensile Load of Material Type 2 Specimens subjected to Accelerated Weathering**

Figs. 5 and 6 show, that the splitting tensile strengths of control specimens can be significantly different within the same test cylinder. This is probably due to non-uniformity of the mix and local crushing that took place at the loaded points in some of the specimens prior to tensile splitting failure. The average of the two adjacent specimens is compared with the weathered specimen in order to look at the effect of the weathering.

Table 2 summarises the loss/gain in tensile strength of the weathered specimens compared with the control specimens.

| *Weathering Agent* | *Material Type* | | | |
|---|---|---|---|---|
| | *A* | | *B* | |
| | *Spec. 2* | *Spec. 5* | *Spec. 2* | *Spec. 5* |
| Sulphuric Acid | -29.2% | -15.7% | -34.3% | -39.7% |
| Sodium Hydroxide | -26.6% | -12.3% | -30.8% | -25.7% |
| Sodium Sulphate | 12.8% | 11.4% | -4.2% | -3.1% |
| Sodium Chloride | 2.4% | -0.8% | 5.2% | -0.7% |

**Table 2. Loss/Gain in Strength after Accelerated Weathering**

Both materials show a similar trend in the results. Effect of both sodium sulphate and sodium chloride solutions is not adverse after continuous wetting and drying for 37 cycles. Any loss or gain in strength shown by the specimens subjected to salt attack is not more than the variability of strengths shown by control specimens. However, it should be noted that both sulphuric acid and the sodium hydroxide have significantly affected the strength of the specimens.

The accelerated agents used in this study, especially the sulphuric acid and the sodium hydroxide, are extreme cases, and unlikely to occur in general applications. However, results obtained reveal that sulphuric acid and sodium hydroxide can be harmful for the polymer concrete materials developed. Further investigations are needed to assess their resistance to mild concentrations of the agents.

## 5 CONCLUDING REMARKS

Two new polymer concrete based materials were developed using industrial wastes. These materials were used to form the basis of a novel construction system. Circular disc specimens of these materials were manufactured in the laboratory so that accelerated weathering and strength tests could be conducted to assess their durability characteristics. Preliminary tests indicate that sulphuric acid and sodium hydroxide significantly affect the tensile strength of these materials and reduce their durability. However, other normally occurring agents such as sodium sulphate and sodium chloride, do not adversely affect these materials. Further investigations are being carried out and will be reported in the future.

## 6 ACKNOWLEDGMENTS

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# Compliance Requirements For Durability Of Masonry Mortar

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**Summary: The durability of masonry mortar is an important property that can significantly influence the performance of a masonry structure. Mortar durability is influenced strongly by both environmental factors and the properties of the masonry units and the mortar. Because of the large number of factors affecting durability, it has become difficult to develop a representative test method to predict the performance of masonry against degradation. There are no codified performance criteria for durability of mortar other than to give prescriptive deemed to satisfy requirements based on experience. The work reported in this paper has developed laboratory and field test methods to assess the service life of masonry mortars subjected to salt attack, abrasion and wind action. The paper also describes performance criteria derived by comparing quantitative performance with qualitative performance.**

**Keywords: Durability, Mortar, Scratch Test, Cycling Test, Performance Assessment**

## 1 INTRODUCTION

Current building practices are changing as industry regulators embrace a performance-based code. Issues such as guaranteeing structural integrity, compliance with new codes and legal responsibility are now fundamental for builders, engineers, building designers, architects and building owners.

In Australia, brick veneer, double-brick or cavity masonry walls are common in residential construction. As medium density construction continues to grow in popularity, so too does the use of masonry as a cladding. A critical aspect of quality masonry is mortar. Mortar in masonry acts as the binding agent between the masonry units as well as accommodating variations in unit dimensions. The mortar must also have adequate workability during laying and adequate strength and durability in service. To meet the Australian Standard AS3700 (Australian Standards 1998) mortar mixes must comply with a performance requirement, and this is usually achieved by complying with specified compositions. If the mortar mix does not comply, it is prone to failure.

Currently there is no accepted test method for assessing the durability of mortar. At present, the Australian masonry code (Australian Standards 1998) contains a performance statement for mortar durability along with some deemed-to-satisfy mortar compositions for various environmental conditions, which are not supported by a testing method or acceptance criteria. No suitable test methods or acceptance criteria are available in overseas standards. Test methods have been developed by other countries for determination of mortar hardness, but they do not adequately reflect the surface hardness required in order to resist mechanical abrasive damage. Although the mechanisms of breakdown are well understood, there is no accepted method for assessing the durability of mortar and the likely lifetime of the joints in normal service.

There is a wide range of variables that must be considered in developing a representative durability test. The type and nature of the test will depend upon its intended use. If it is to compare the performance of various cement, sand or additive types, the test does not necessarily need to produce the site conditions. Developing a test method and relating its results to field performance is a large task. It would require long term monitoring of the durability performance of masonry panels located in severe exposure conditions in parallel with laboratory experiments. Another alternative is to attempt to create in the laboratory an environment and test specimens that represent as closely as possible the site conditions. An accelerated test would then be able to give a reasonably quick indication of the long term performance. However, more importantly, what is required is a simple apparatus that can assess the durability performance of in-situ walls.

This paper highlights some of the common failures of mortar in masonry structures. Development of field and laboratory test methods to assess the mortar degradation is described, and a correlation between the two test methods has been established. A suitable performance criterion for mortar is suggested.

## 2 MODES OF DETERIORATION

Mortar durability is influenced by many factors, both internal and external to the mortar joint (Bowler 1993, Ludwig 1980 and Harrison & Gaze 1989). In contrast to many countries, most of the population centres in Australia are not subjected to freeze-thaw cycles. However, many cities are located along the coast and therefore severe exposure conditions prevail such as salt attack and wind action.

Salt attack is predominantly considered more of a physical nature than a chemical action. In this mechanism, crystallisation of the salts in the pores of the mortar may result in its disruption owing to the pressure exerted by the salt crystals. As a result, the denser the material, the less is the likely deterioration.

Poor batching practices on site can often result in a cement content lower than that specified. The level of cement in the mix will directly influence the weathering rate of a mortar joint by wind action or abrasion.

With the continued development and increasing use of blended cements and chemical admixtures, there is concern that the durability of mortars made with these products may be affected.

## 3 ASSESSMENT OF DURABILITY

Representative laboratory and field tests for durability are required to accelerate the most likely mechanisms of failure due to environmental effects, so that an assessment can be performed in a reasonable time. Since no universal test for mortar durability exists, two possible test methods have been previously proposed (Samarasinghe & Lawrence 2000 & 1998) for possible development and standardisation. Those tests will allow assessing the adequacy of in-situ mortar to resist environmental degradation. Such test methods would also enable the performance of comparative studies of the durability performance of various mortar types with different compositions.

### 3.1 Laboratory Test Method

There is much common ground between concrete and masonry technology in relation to the durability of cements. Many investigators have studied this problem, mostly in relation to the performance of concrete rather than masonry. Tests usually consist of some form of soaking in an aggressive medium and monitoring the progressive deterioration of the specimen. The tests developed by the authors were similar in principal, and adopted the RILEM test for durability assessment of hardened mortar under freeze-thaw conditions (RILEM 1998) with some modifications. A controlled regime of soaking (in 5% sodium chloride) and drying cycles on mortar tablets, prepared from mortar joints cast between bricks, have been used for simulating the mechanical breakdown of hardened mortar particles.

A wide range of methods has been suggested by various investigators to assess the degree of deterioration of specimens subjected to accelerated weathering. The most commonly used techniques are physical appearance, change in volume (or dimensions), change in weight and reduction in strength. However, in masonry joints, most of the durability problems occur at the surface, and therefore strength measurements would not reflect the actual level of degradation. Hence, the percentage change in weight of the specimens was considered as the indicator of degradation.

Typical illustrations of the results are shown in Figures 1 and 2. The '2% weight loss' criterion was selected as the suitable index for the durability assessment of mortar. This is measured as the cycle number at which the specimen weight drops to 2% below its original weight. As seen from Figures 1 and 2, the test method is quite capable of identifying the effect of different parameters (such as cement content, cement type, and also sand type and brick type) that can influence the durability of mortar.

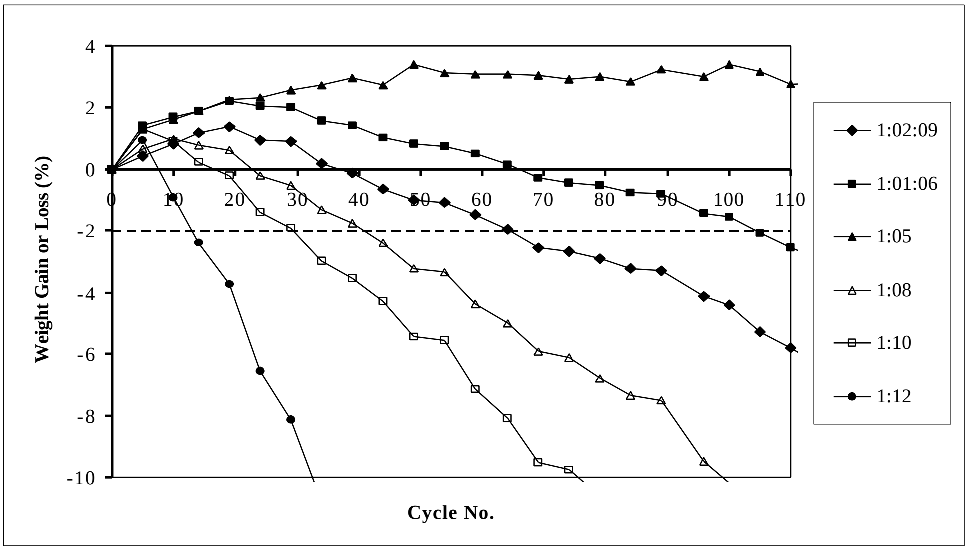


**Figure 1. Typical Weight Curves for Portland Cement Mortars in 5% NaCl**

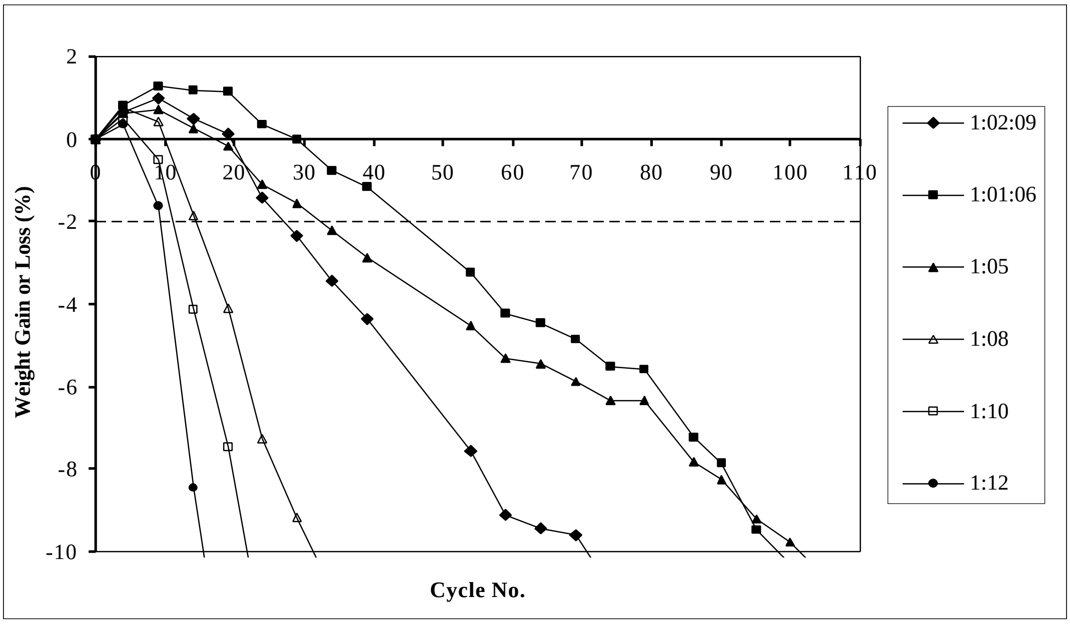


**Figure 2. Typical Weight Curves for Blended Cement Mortars in 5% NaCl**

### 3.2 In-situ Test Method

In the absence of a known precedent for this type of test, a prototype apparatus for a scratch test was designed and built. The principle involved is that a fixed force is applied through a spring to a probe with an abrasive end. The probe is turned through a fixed number of turns and the indentation into the mortar surface is measured with a depth micrometer. It is possible to vary the severity of the test by changing the form of the probe, the spring force and/or changing the number of turns used to make a measurement.

A series of scratch tests was carried out on brick-mortar couplets made with various mortar compositions and the indentation after 5, 10, 15 and 20 turns of the scratch probe was measured. Both scratch tests and cycling tests for comparison on the same specimens were considered impractical. The scratch test results are influenced by the confinement effects of the masonry units on either side of the joint during test, and this confinement is not present for the mortar slabs prepared for the cycling test. Therefore, in order to establish a correlation between the two test methods, specimens were prepared in parallel for the two methods.

A typical illustration of the test results is shown in Figure 3. For establishing a performance criterion and exploring correlation with the salt-cyclic test it is necessary to define an index of performance. Two possibilities have been explored; initial slope of the curves (penetration versus number of turns) and the depth of penetration after the first five turns. Examination of these indices showed that, while there was variation in the results, both appeared useful to represent the penetration results and they both appeared to correlate with the cement content in the mix. However, the index of average penetration for the first five turns (mm) was chosen as the best index for this test.

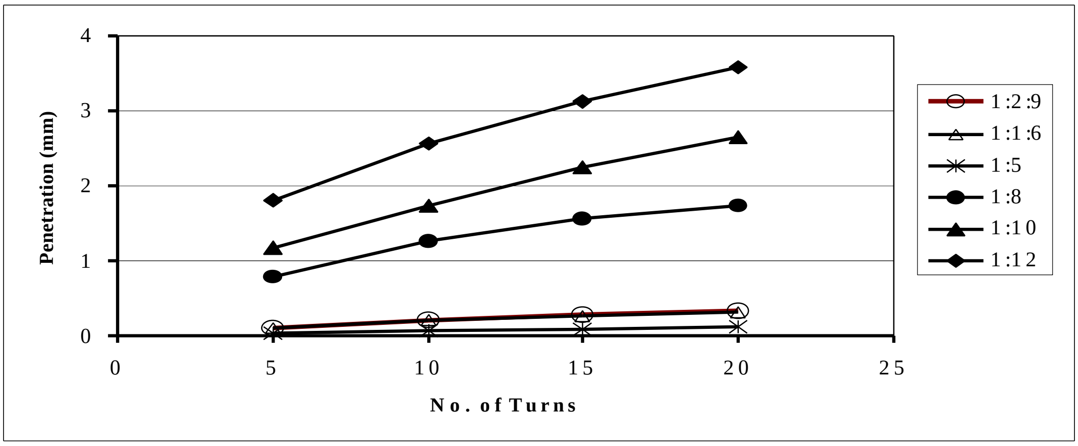


**Figure 3. Typical Relationships Between Penetration Depth and Number of Turns for the Scratch Test Correlation between the Two Test Methods**

The indices of performance of the two test methods are plotted to a logarithmic scale in Figure 4. The index of performance for the salt cycling test is the cycle number at which two percent weight loss of the test specimen occurs, whereas for the in-situ scratch test, it is the rate of penetration within the first five turns. Correlation between the cyclic test and the scratch test is good; indicating that the scratch test used in the field is likely to measure the same property as the cyclic test used in the laboratory.

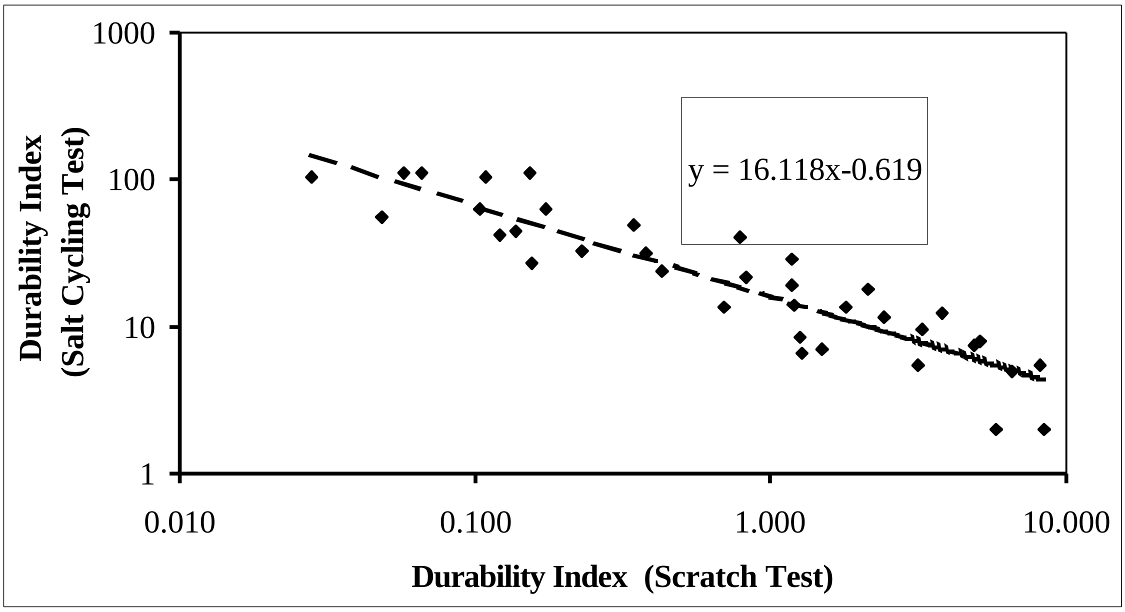


**Figure 4. Correlation of Scratch Index with Weight Loss Index**

## 4 CALIBRATION OF TEST METHODS

It is relatively easy to identify the relative effects of different parameters in the laboratory, which affect the durability performance. However, the major task is assessing the test results and relating them to wall performance. Assessing wall performance can be subjective, and therefore, some calibration testing would be necessary for this purpose.

Calibration of the scratch test index was conducted using exposure tests on specimens prepared under laboratory conditions.

Four-course prisms were prepared with 1:5, 1:8, 1:12 and 1:15 mortar mixes and placed on exposure racks after 14 days curing time. Three types of cements (GP, GB (SB), GB (FAB)) and two types of sand (Sydney Sand and Fatty Sand) were used with each of the mixes selected. One prism was built for each combination of materials. The prisms were orientated in the North-South direction with greatest sunlight exposure on the northern face and severe wind and rain attack on the southern face.

Scratch tests were carried out on the weather exposed mortar joints using the scratch tool after 22 months. Observations were also made about the appearance of the joints after the weather exposure, and they were gently rubbed with a finger along the joint to make a qualitative assessment about it's the abrasion resistance. Durable mortar should not have any loss of ingredients under a gentle abrasive force.

The abrasive resistance of the joints was categorised into four levels.

1. No dislodging of mortar ingredients – Level A.
2. Slight dislodging of ingredients, but stops after 2-3 rubs – Level B.
3. Moderate dislodging of ingredients in which small amounts of particles loosened at every rub – Level C.
4. Severe dislodging of ingredients with large quantities of particles loosened at every rub – Level D.

Both moderate and severe dislodging of ingredients of mortar are not acceptable, and people tend to complain about such masonry. All the penetration data were pooled together, irrespective of the mortar mix, and investigated for the correlation between the five-turn penetration depth and the Rub Test Grade. The average penetration depths corresponding to each rub-test grade were plotted as shown in Figure 5.

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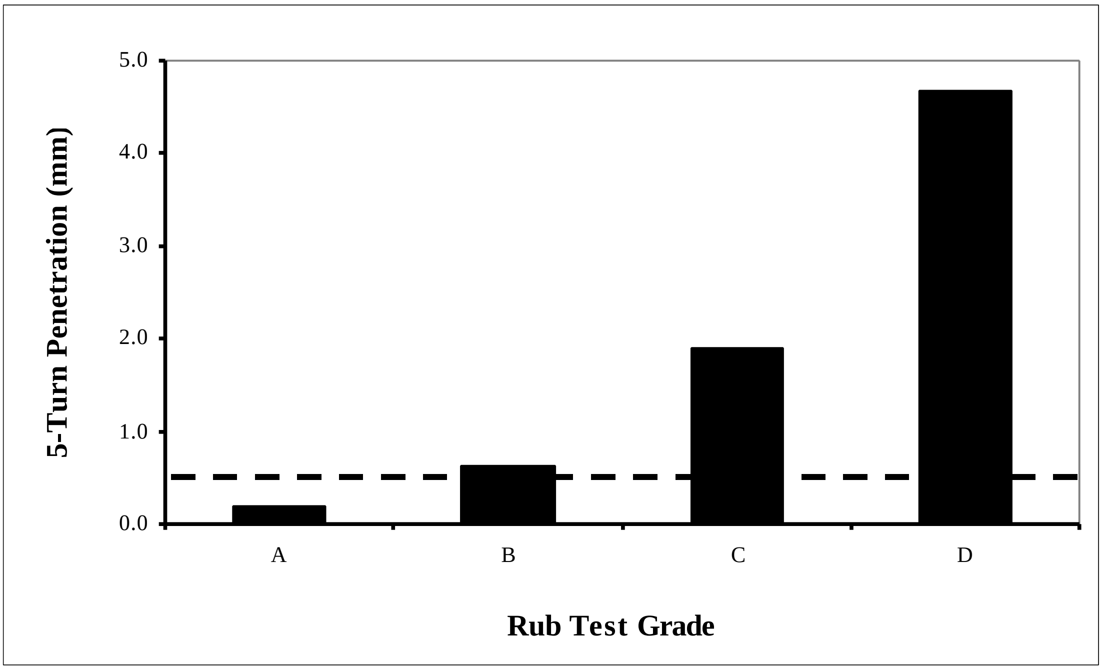


**Figure 5. Correlation between the Rub Test and the Scratch Test**

Comparing the results of the finger abrasion test and the scratch tool penetration, it could be suggested that any mortar with penetration greater than two millimetres is not acceptable. When the penetration is greater than two millimetres, mortar is more likely to be damaged under a mechanical abrasive force. This criterion was later verified with field data.

Therefore, the relationship between the two percent weight loss criterion and the five-turn penetration of the scratch tool can be used to establish the acceptance criterion for the salt-cyclic tests. The relationship established by correlating the results from the salt-cyclic test and the scratch test is as follows.

$N = 16.118\ D^{-0.619}$

Where,

N = Cycle Number at which two percent weight loss occurs in the NaCl cyclic test

D = Scratch Tool Penetration after 5 turns (mm)

Hence, when D = 2 mm,

N = 10.5

Therefore, if the cycle number at which the tablet weight reached two percent below its original weight in the NaCl cyclic test is less than 11, such a mortar is not considered durable.

## 5 CONCLUSIONS

A salt-cyclic test has been developed, using mortar tablets cast between bricks. Results have shown that five percent sodium chloride solution causes breakdown of the specimens in a reasonable time. The test method is capable of distinguishing the resistance to degradation of mortars with a range of different compositions. The cycle number for two percent weight loss has been chosen as the most useful index of performance for this test.

A controlled scratch provides a useful measure of the durability resistance of the mortar. The average penetration per turn for the first five turns of the scratch device has been selected as the index of performance for this test. Tests on a wide range of mortars have shown that the test is capable of identifying mortars with a range of different mix proportions.

The salt-cyclic test and the scratch test give measures that correlate well with each other and are therefore measuring the same property. Laboratory exposure tests have been used to establish a suitable criterion for the performance assessment of mortar.

## 6 ACKNOWLEDGMENTS

The Cement and Concrete Research Association of Australia and the CSIRO supported the research.

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# Development Of RILEM Durability Test Method For Curtainwall Sealants

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**Summary: The paper discusses the work carried out over the past decade within ISO TC59/SC8 and RILEM TC139-DBS committees towards the development of a durability test standard for sealants. In 2001, RILEM TC139-DBS published a RILEM Technical Recommendation (RTR) on a durability test method for curtain wall joint sealants. The paper discusses the development of this test method as well as results obtained in the evaluation of sealants. Results of initial evaluations indicate that the test method is able to differentiate between products with regard to their resistance to accelerated ageing and mechanical cycling. The type of failure and the changes in surface appearance observed during the test regime are similar to those observed in actual service conditions. Sealant degradation resulting from durability cycles without fatigue cycling was slower than had been anticipated for most sealant systems.**

**Keywords: Durability, Sealant, Test Method, Curtainwall, RILEM.**

## 1 INTRODUCTION

Over the past two decades, the sealants industry has undergone rapid technological and structural changes. On the one hand, advancements in technology have enabled the launch of a multitude of new sealant products based on novel polymers, cure chemistries and formulations. On the other hand, increasing competitive pressure and customers that are more demanding have required shorter product development cycles. Unlike the well-established sealants, which have been sold for more than twenty years based on the same formulation, the new sealant products do not have documented long-term performance histories. At present, generating a reliable performance history for a new sealant product still requires long-term outdoor testing and extensive in-service field evaluations. Attempts at avoiding these tasks, by employing various forms of short-term laboratory-based ageing tests, have had limited success and are viewed with suspicion by construction specifiers, mainly because of the lack of an established correlation with actual in-service performance of sealants.

The sealants industry, therefore, urgently needs a method for generating long-term performance data rapidly and with assured reliability. Accelerated laboratory ageing experiments are the most promising method for acquiring durability information within the shortest possible time; however, a methodology for conducting and interpreting these experiments needs to be developed that improves the predictive value of this technique.

The need for improved longevity of sealed joints is well recognised. Work towards an accelerated durability test method was started in 1989 within the International Standardisation Organisation Committee ISO TC59/SC8 (Work Group 6). Later, in 1994, the activity was transferred to RILEM TC139-DBS Durability of Building Sealants, when the ISO committee realised that the task was too complex to be completed within the five-year time frame allowed for the development of an ISO standard. The RILEM committee has now developed a Technical Recommendation (RTR) (RILEM 2001), which will be considered by ISO for the development of a future durability test standard. The purpose of this technical recommendation is to provide a framework for assessing the effects of cyclic movement and artificial weathering on curtain-wall sealants in a laboratory-based procedure.

## 2 DEVELOPMENT OF A DURABILITY TEST METHOD FOR SEALANTS

During their entire service life, joint seals are exposed to cyclic mechanical strain and environmental degradation factors. Cyclic joint movement, sunlight, temperature variations (heat, cold) and moisture (water) are considered to be the primary environmental and service degradation factors leading to sealed joint failure. Weatherproofing joint seals in building façades are exposed to frequent cyclic movements. This joint movement imposes cyclic mechanical strain on the seal, which, depending on the exposure conditions and the construction design, can vary substantially in rate and amplitude.

Work on the durability test method started with the premise that the accelerated testing regime should incorporate only the following four key ageing factors: solar radiation, moisture, temperature and joint movement. With this limitation, the committee realised the impossibility of incorporation all possible combinations of weathering and service factors into a single laboratory-based testing regime. Weathering and service factors, which the committee specifically did not intend to address, were:

- Dirt accumulation
- Acid rain
- Cleaning solvents
- Microbial growth
- Incompatibility with other building materials

Joint movement is the preponderant service factor in the ageing of sealants, since it has a damaging effect both while the sealant is curing and after completion of the cure. Depending on the type of joints, sealants are exposed to various types and degrees of movement. Joint movements are induced by various factors, such as:

- Thermal expansion/contraction of construction components
- Shrinkage of construction components
- Settling of buildings
- Wind loads
- Service loads

Some of these factors may occur simultaneously, exposing the sealed joint to a rather complex, stochastic, often three-dimensional movement pattern of overlapping shear and extension/compression movements. Such complex movement patterns are difficult to simulate and the analysis of its effect on the sealant properties is an even more challenging task. Therefore, for the purpose of developing an accelerated durability test standard, the RILEM TC139-DBS committee decided to consider only the following three simple movement types:

- One single large movement followed by essentially no further movement
- One large movement followed by some cyclic movement
- Cyclic movement (in shear or tensile)

The above movement types can be roughly correlated with the movement patterns occurring in settlement joints, settlement joints with some movement, and glazing or weatherproofing joints, respectively.

Considering the frequency with which sealants are being used for the various applications, expansion and glazing joints represent the majority of sealant usage. Therefore, the development of a test method based on cyclic movement was prioritised by the committee. Obviously, the type of movement (tensile or shear) as well as the amplitude and rate of movement still depend on the type of application. A joint in a monolithic concrete façade, for instance, is likely to see slower and smaller movements than a joint in an aluminium curtain wall façade. For simplification, the committee had to narrow the scope of the initial test method again, and it decided to initially focus on cyclic tensile movements.

Even joints that are predominately exposed to cyclic tensile movement still show a rather complex movement pattern (see Hutchinson *et al*. 1999 and literature cited therein). However, based on experience with the development of movement capability tests (ISO 1989, Wolf 1999 and literature cited therein), it was assumed that the actual joint movement could be approximated by using idealised cyclic movements, such as trapezoidal, V-shaped, or sinusoidal movements. Research, comparing the performance of sealants exposed to various waveforms, suggested that trapezoidal movement waveforms provide the best correlation with simulated in-service performance (Jones *et al*. 1999). Therefore, trapezoidal movement waveforms were selected for the accelerated durability test method.

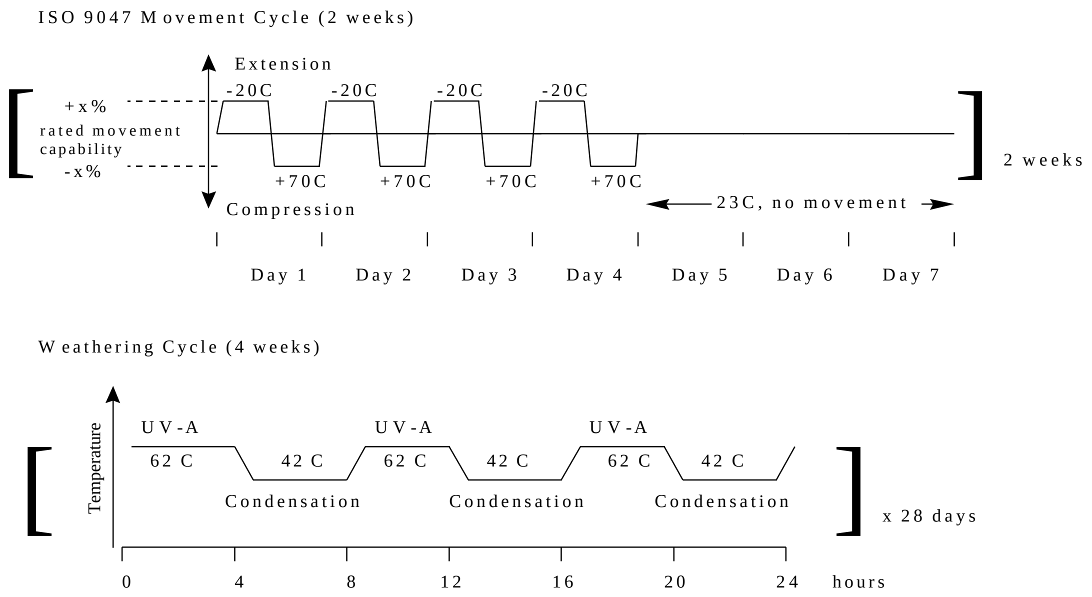


**Figure 1. Initial proposal of an accelerated durability test method (1993)**

A further narrowing of the scope of the initial test standard occurred when the committee decided to consider only elastomeric sealants, as defined by the ISO 11600 classification scheme (ISO 2000). Since ISO 9047 (ISO 1989) is the relevant test standard for determining the movement capability of cured elastomeric sealants, it was the most obvious candidate to consider as movement cycle for the future durability test standard. A first proposal of a test method (ISO 1993) suggested exposing the cured sealants to consecutive durability cycles. Each durability cycle consisted of a two week movement cycle as defined in ISO 9047 and a four week weathering period consisting of repeated cycles of four hours fluorescent lamp UV-A exposure followed by four hours moisture condensation (see Fig. 1). The proposal suggested that the durability cycles should be continued, until failure occurred, which was to be detected by visual inspection of the specimens after each durability cycle (see Fig. 2). The procedure proposed for the inspection was to extend the specimens to their rated movement capability and to place the extended specimens over a light.

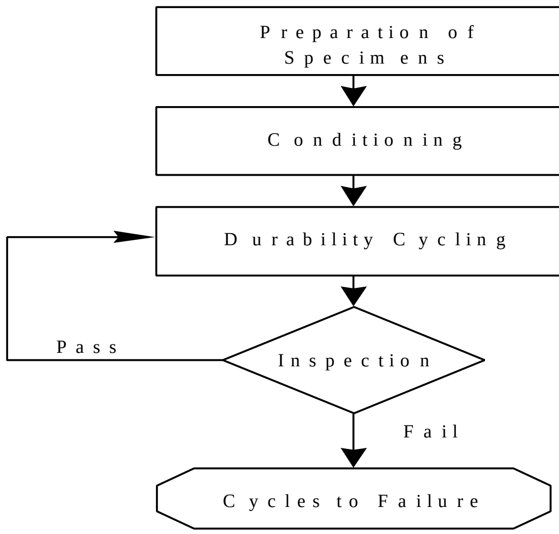


**Figure 2. Proposed concept for accelerated durability testing (1993)**

Much discussion revolved around the question of how the sealants should be conditioned prior to exposing them to the durability cycles. The proposal to use the same conditioning methods as defined in ISO 11600 was widely supported, based on the assumption that this approach allowed direct comparison of the result of the durability cycles with the 'initial' properties as determined by the other test methods referenced in ISO 11600. On the other hand, previous testing had clearly demonstrated the negative impact of mechanical cycling during cure on the properties of sealed joints (Jones & Lacasse 1999, Wolf 1999 and literature cited therein). Therefore a 'dynamic cure' procedure was proposed (Jones & Hutchinson 1998), in which the sealed joint is mechanically cycled while the sealant cures. Weathering of a sealed joint during cure also has an effect on its 'initial' properties (Beech & Turner 1983); the effect is especially noticeable for those weathering factors that either accelerate or retard the cure of the sealant. However, the effect appeared to be smaller than that of joint movement. The committee, therefore, agreed that it would be sufficient to consider only the effect of movement during cure to establish the initial reference state.

## 3 SUMMARY OF THE RTR DURABILITY TEST METHOD

The RILEM Technical Recommendation "Durability test method - Determination of changes in adhesion, cohesion and appearance of elastic weatherproofing sealants for high movement façade joints after exposure to artificial weathering" (RILEM 2001) specifies a laboratory procedure for determining the effects of cyclic movement and artificial weathering on laboratory cured, elastic weatherproofing joint sealants (one- or multi-component) for use in high movement building façade applications.

In this method, test specimens are prepared in which the sealant to be tested adheres to two parallel contact surfaces (substrates). Sealant specimens are conditioned either statically (no movement) or dynamically (exposed to cyclic movement). The conditioned sealant specimens are then exposed to repetitive cycles of artificial weathering (light, heat and moisture) and cyclic movement under controlled environmental conditions (degradation cycles). Weathering is carried out for eight weeks (default value) in an artificial weathering machine. This is followed (optionally) by rapid mechanical fatigue cycling (default: 200 cycles). The specimens are then exposed to two thermo-mechanical cycles as defined in ISO 9047 (section 8, first week), using the full amplitude suggested as the movement range of the sealant under test.

After completion of each degradation cycle, the specimens are extended to their full rated extension and held there as the sealant beads are visually examined for changes in appearance, cohesion and adhesion. The depth of any cohesive or adhesive flaw is determined according to the rules provided in ISO/DIS 11600 and the general condition of the sealant is reported. The weathering exposure, the cyclic movement, and the examination for failures constitute a degradation cycle and the degradation cycle is repeated as often as desired to achieve a certain exposure. A schematic representation of the test procedure is shown in Fig. 3.

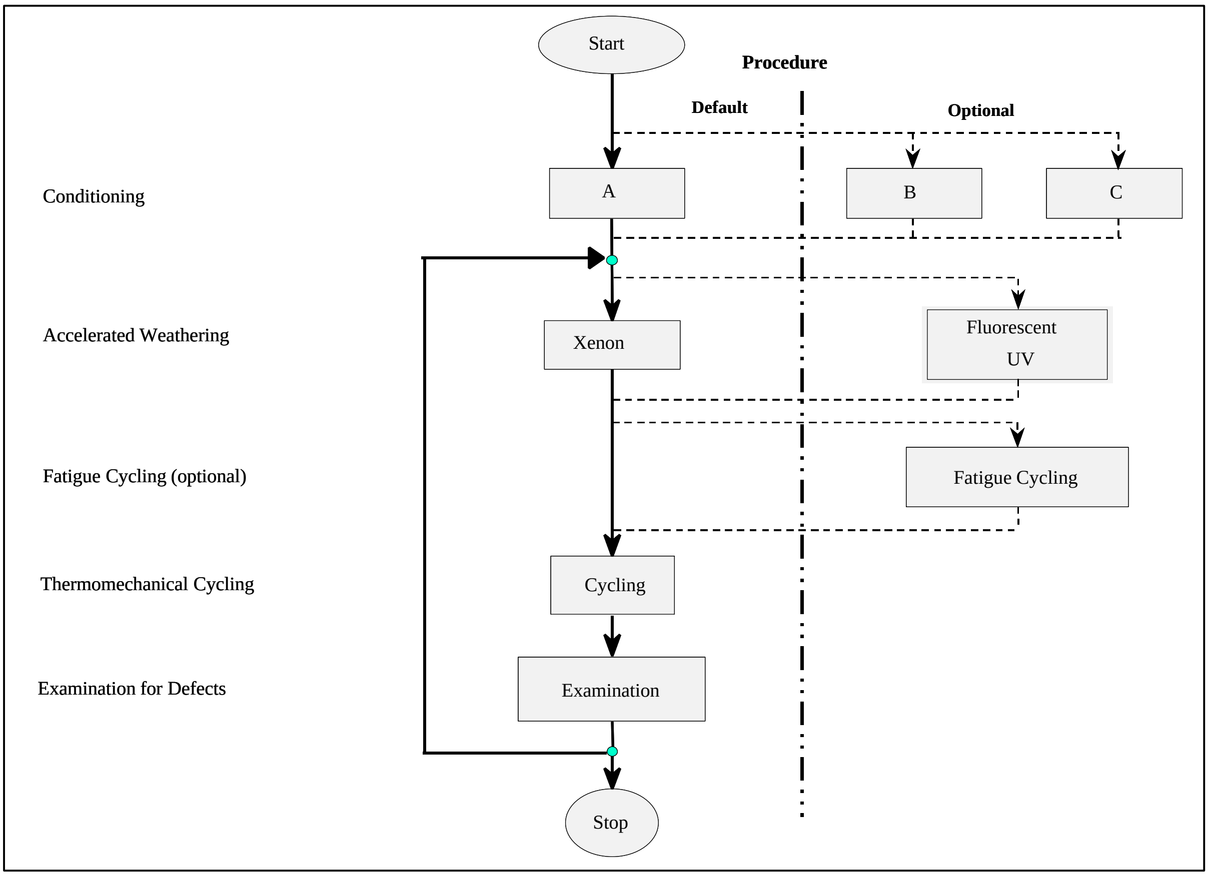


**Figure 3. Schematic representation of RILEM RTR test procedure**

Default test parameters and, for some procedures, alternative options are defined in the technical recommendation (see Table 1). In cases of dispute, the default method is the reference method. The experimenter is allowed to deviate from the default values for the following test parameters, however, is required to highlight any deviation from the default values in the test report:

a) Substrate – default: anodised aluminium as specified in ISO 13640 (ISO 1996)
b) Support dimensions – default: 75mm x 12mm x 6mm as specified in ISO 8339 (ISO 1984)
c) Conditioning method (A, B or C) – default: A as specified in ISO 11600
d) Artificial light source (xenon-arc, fluorescent UVA-340 lamp) – default: xenon-arc as specified in ISO 4892-1-3 (ISO 1998a)
e) Weathering procedure: duration of artificial weathering, type of moisture exposure (spraying or immersion), the temperature of light exposure, the temperature of moisture exposure, the timing of light and moisture/water cycle – default values are specified for xenon arc/water-spray, xenon-arc/water immersion and fluorescent UVA-340/water-spray weathering;
f) Rapid fatigue cycling (optional): inclusion of fatigue cycling, amplitude and duration (number of cycles) of fatigue cycling, default: 200 cycles – as specified in JIS A 1439 (JISC 1997);
g) Thermo-mechanical cycling (ISO 9047 type): amplitude and duration (number of cycles) – default values are specified in the test procedure.

**Table 1. Overview of default and alternative choices of key test parameters**

| *Procedure* | *Default* | *Alternative option* |
|---|---|---|
| Conditioning | A | B or C |
| Movement parameters for Conditioning C | 1 cycle/day<br>±7.5% amplitude<br>2.4 hour periods<br>trapezoidal waveform<br>extension/compression rate 70±20 mm/min<br>1st stroke in tension | |
| Accelerated weathering | Xenon arc light and water spray<br>102 min. light at 65±5°C black standard thermometer and 60 ±10 % rel. humidity<br>18 min. light with water spray<br>672 cycles (8 weeks)<br>or Xenon arc light and water immersion<br>102 min. light at 65 +//-5°C black standard thermometer; 18 min. light during water immersion<br>672 cycles (8 weeks) | Fluorescent ultraviolet radiation and water-spray (FL/UVA-340 lamps)<br>8h UV at 65±5°C black panel thermometer<br>4h UV with water spray<br>112 cycles (8 weeks) |
| Fatigue degradation | ---------------------------------------------- | Isothermal cycling |
| Movement parameters for isothermal cycling | 5 cycles/min at rated movement capability amplitude<br>200 cycles total | |
| Thermomechanical cycling | 2 cycles at rated movement capability amplitude (ISO 9047, 1st week) | |

The use of this method is intended to induce property changes in sealants associated with typical end use conditions, i.e. to simulate a natural weathering environment of sealants installed in curtain-wall joints exposed to high joint movement. Exposures are not intended to simulate the deterioration caused by localised environmental conditions, such as atmospheric pollution, biological attack or salt-water exposure. It should be noted that the use of this method as a predictor of the service life of a sealed joint for a given climate and location and on a given building has not been demonstrated yet. In the future, it is hoped that the test parameters can be linked with specific climatic zones and actual exposure conditions on site.

## 4 RESULTS OBTAINED WITH RTR DURABILITY TEST METHOD

### 4.1 Oxford Brookes University study (1999-2000)

As the committee started to converge towards the final test method in 1999, the importance of evaluating the behaviour of a set of sealants under comparative conditions became apparent in order to validate the proposed durability test method. A first study was completed by Oxford Brookes University (Jones *et al.* 2001), in which a total of fifteen different high quality sealant products, supplied by twelve manufacturers, were tested, including five silicones, four polyurethanes, three silicon-modified polyethers, two polysulfides, and one solvent-borne, silicon-modified acrylic. Anodised aluminium as specified in ISO-DIS 13640 (ISO 1998b) was used as a substrate material. Primers were used on the anodised aluminium substrates as recommended by the manufacturers.

All test specimens were subjected to four durability cycles. A durability cycle consisted of the joints being subjected to eight weeks of artificial weathering in a fluorescent UV/condensation device followed by two full ISO 9047 movement cycles (4 days). Following the fourth durability cycle, it became apparent that the durability cycles did not accelerate sealant degradation as quickly as originally thought. It was therefore decided to incorporate 1000 fatigue cycles with amplitude of ±25% at a rate of 5 cycles/minute into the experimental programme following the fourth durability cycle. The results from the fatigue cycling were intended to give insight into the possibility of including fatigue into later draft versions of the durability test method. Figure 4 shows photos of representative test specimens at the discontinuation of the test.

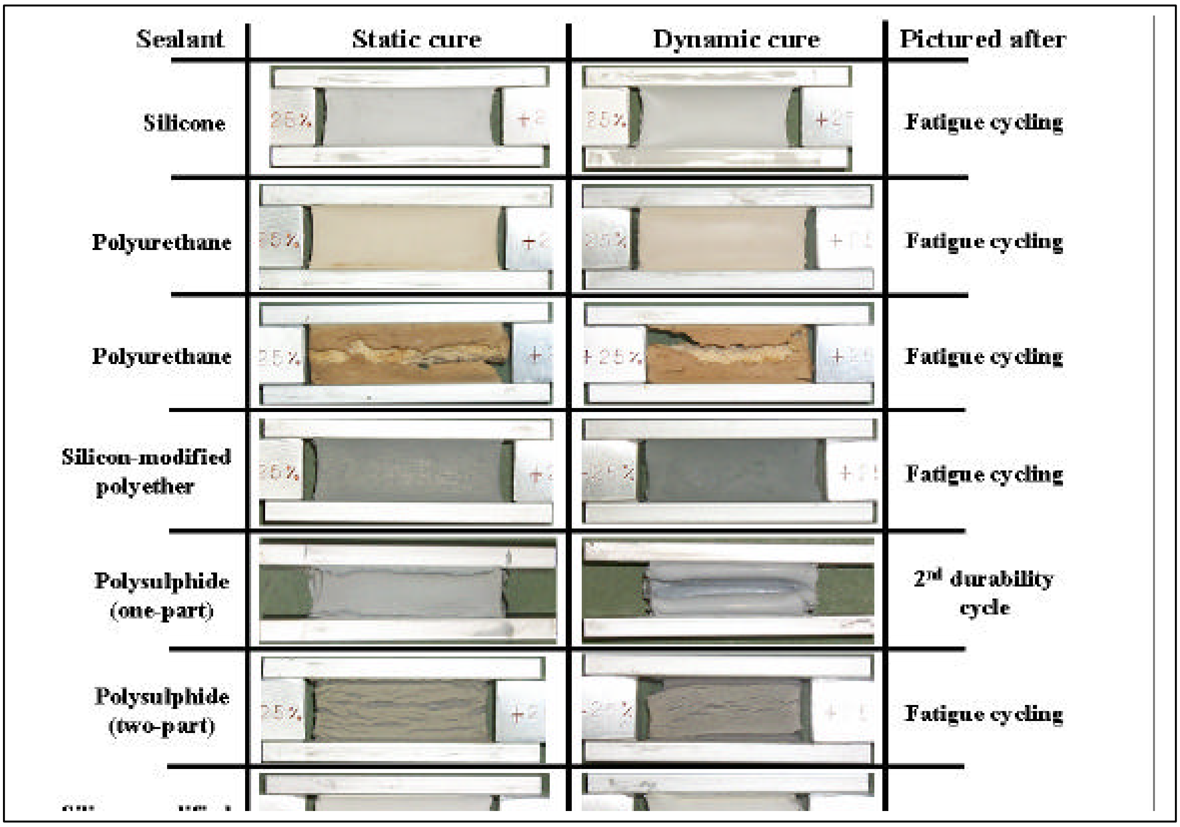


**Figure 4. Representative test specimens at discontinuation of test**

Exposing the silicone sealants to the durability cycles showed no effect for the first three cycles. After the fourth durability cycle, some silicone sealants showed a minor loss of adhesion, primarily at the ends of the joints, where stresses are highest. For these sealants, further loss of adhesion occurred during the fatigue cycling phase, but it proceeded at a different rate for each sealant. In general, no substantial differences were observed in the performance of dynamically and statically conditioned specimens, except for slight bead deformation resulting from the dynamic conditioning. The surface appearance of the silicone sealants tested was not affected by the durability cycles; no cracking, crazing, or visible discoloration occurred.

Large differences in the performance of the tested polyurethane sealants were observed. Except for taking a slight compression set, a one-part polyurethane sealant exhibited few signs of deterioration following the durability cycles. Another one-part polyurethane sealant lost adhesion at the ends of the joints, discoloured and developed surface crazing. The three-component polyurethane sealant showed severe discolouration soon after the first durability cycle and extensive blisters and cohesive cracks developed after the second cycle. The cohesive cracks propagated into the sealant and finally led to complete cohesive failure after the fatigue cycling.

The silicon-modified polyether sealants tested were little affected by the first two durability cycles. Surface crazing was observed for one sealant after the second cycle; the other two sealants started to craze after the third cycle. Two sealants survived four durability cycles, exhibiting no or only minor adhesion loss. All silicon-modified polyether sealants started to fail adhesively during the fatigue cycling.

The one-part polysulfide sealant cured very slowly and remained incompletely cured even after being exposed to three durability cycles. Due to the incomplete cure, the sealant developed major folds in the sealant bead, which further developed into cohesive cracks that finally resulted in complete cohesive failure after the second and third durability cycle for the dynamically and statically cured specimens respectively. Clearly, this sealant was not able to withstand the large and rapid movements that may occur in a curtain wall joint, due to its slow cure. The two-part polysulfide sealant performed better, although cracks developed on the bead surface after the second durability cycle, which continued to propagate into the depth of the sealant during the following cycles.

The silicon-modified polyacrylate sealant performed better than the other silicon-modified sealants. Very little deterioration was observed for the first three durability cycles. After the fourth durability cycle, some loss of adhesion occurred along the edge and the ends of the joints. The area of adhesion loss increased because of the fatigue cycling.

In general, the Oxford Brookes study found that the RILEM durability test method was able to differentiate between products with regard to their resistance to accelerated ageing and mechanical cycling. The type of failure and the changes in surface appearance observed during the test regime were similar to those observed in actual service conditions. The study confirmed that the initial modulus of a sealant is not a good indicator of the performance in the durability test; apparently more important are the property changes that occur because of the ageing (hardening, reversion, etc.). The dynamic conditioning procedure reduced joint durability for some of the tested products, but increased it for others. Sealant degradation resulting from the durability cycles was slower than had been anticipated for most sealant systems. The fatigue cycling clearly accelerated sealant degradation. It was therefore proposed to include fatigue cycling as part of each durability cycle. The RILEM TC139-DBS committee followed this recommendation and included the fatigue cycling as an option in the final technical recommendation.

### 4.2 Tokyo Institute of Technology study (2001)

A further study was initiated by the Japan Sealant Industry Association (JSIA) after the RILEM Technical Recommendation (RTR) on the durability test method had been finalised. This study, carried out by the Tokyo Institute of Technology in Yokohama (Takana & Miyauchi 2002), has currently completed two durability cycles and will continue for another durability cycle. The study comprises eleven sealants, two silicones, two silicon-modified polyethers, two polysulphides, two polyurethanes – each as one- and two-part products – one two-part silicon-modified polyisobutylene, one two-part urethane cure acrylic, and a one-part water-borne acrylic. The two-part polysulphide sealant is based on a new type of polysulphide/polyether/polysulphide co-polymer and employs isocyanate cure chemistry. Test specimens were prepared using anodised aluminium and mortar as specified in ISO-DIS 13640 as substrate materials; primers were used for all sealant/substrate combinations as recommended by the manufacturers. The two-part polyurethane, the two-part urethane-cure acrylic and the one-part water-borne acrylic were also evaluated after painting the sealant surface with a highly elastic paint. All sealant specimens were conditioned according to method A. Weathering was conducted in a fully automated weathering machine using a Xenon-lamp light source. Durability cycles were carried out with and without inclusion of the 200-cycle fatigue ageing.  Table 2 displays a summary of the interim results after the second durability cycle.

**Table 2. Interim results of Japanese study after second durability cycle**

| *Sealant* | *Substrate* | *Surface painting* | *Fatigue cycling* | *Chalking*[1] | *Crazing*[2] | *Test result* |
|---|---|---|---|---|---|---|
| SR-1 | Aluminium | No | Without | 1 | 0 | Pass |
| | | | With | 1 | 0 | Pass |
| SR-2 | Aluminium | No | Without | 1 | 0 | Pass |
| | | | With | 1 | 0 | Pass |
| MS-1 | Aluminium | No | Without | 2 | 0 | Fail (1st cycle) |
| | | | With | 2 | 0 | Fail (1st cycle) |
| MS-2 | Aluminium | No | Without | 3-4 | 4-5 | Pass |
| | | | With | 3-4 | 4-5 | Pass |
| PS-1 | Aluminium | No | Without | 3 | 3-4 | Pass |
| | | | With | 3 | 3-4 | Pass |
| PS-2 | Aluminium | No | Without | 4 | 5 | Pass |
| | | | With | 4 | 5 | Fail (2nd cycle) |
| PU-1 | Aluminium | No | Without | 2 | 0 | Pass |
| | | | With | 2 | 0 | Pass |
| PU-2 | Aluminium | No | Without | 5 | 5 | Fail (1st cycle) |
| | | | With | 5 | 5 | Fail (1st cycle) |
| IB-2 | Aluminium | No | Without | 1 | 2 | Pass |
| | | | With | 1 | 3-4 | Pass |
| PU-2 | Mortar | No | Without | 5 | 5 | Fail (1st cycle) |
| | | | With | 5 | 5 | Fail (1st cycle) |
| | | Yes | Without | - | - | Pass |
| | | | With | - | - | Fail (1/3) (2nd cycle) |
| UA-2 | Mortar | No | Without | 2-3 | 5 | Pass |
| | | | With | 2-3 | 5 | Pass |
| | | Yes | Without | - | - | Pass |
| | | | With | - | - | Pass |
| AC-1 | Mortar | No | Without | 3 | 1 | Pass |
| | | | With | 1 | 1 | Fail (2nd cycle) |
| | | Yes | Without | - | - | Pass |
| | | | With | - | - | Fail |

[1] Chalking was assessed according to ISO 4628-6 (ISO 1990)
[2] Crazing was assessed according to ISO CD 4628-4 (ISO 1994)

Table legend: XX-1, XX: chemical type, –1 or –2: one- or two-part
SR: Silicone, MS: Silicon-modified polyether, PS: Polysulphide, PU: Polyurethane,
IB: Silicon-modified polyisobutylene, UA: Urethane-cured acrylic, AC: water-borne
Acrylic; Chalking: 1 (none) – 5 (strong); Crazing: 0 (none) – 5 (strong)

The silicone sealant specimens survived the first and second durability cycles without any signs of degradation – no cracks, no crazing, no chalking and no loss of adhesion were observed. Both silicon-modified polyether products showed signs of moderate chalking after the first durability cycle. The one-part silicon-modified polyether sealant took a strong compression set during the ISO 9047 thermo-mechanical cycle (3 mm set based on 12 mm joint width after 25% compression), resulting in adhesive failure during the first durability cycle. However, for this sealant, there is a possibility of an experimental mistake during the preparation of the test specimens. In spite of exhibiting moderate to strong chalking and crazing as well as a 2 mm compression set, the two-part silicon-modified polyether sealant passed both durability cycles. The one-part polysulphide sealant also passed both durability cycles, despite a 2.5 mm compression set and signs of moderate chalking and crazing. Showing strong chalking and crazing, the two-part polysulphide sealant passed both durability cycles, when no fatigue cycling was included, however, two out of three test specimens failed the second durability cycle, when fatigue cycling was included. The one-part polyurethane sealant passed both durability cycles and showed some chalking, but no signs of crazing, regardless whether or not the sealant was exposed to fatigue cycling. The two-part polyurethane sealant chalked and crazed badly already after the first durability cycle. Due to the depth of the surface cracks (2-3 mm), this sealant failed the ISO 11600 pass criterion both on aluminium and mortar substrates. When the same sealant was painted with a highly elastic paint prior to weathering, the sealant passed the first durability cycle on mortar substrates with and without fatigue cycling without problems, but failed the second durability cycle, when fatigue cycling was included. The silicon-modified polyisobutylene sealant passed both durability cycles, but showed signs of moderate crazing after the second cycle. The urethane-cured acrylic sealant passed both durability cycles, with and without fatigue cycling, regardless whether it was painted or not. Without surface paint, it developed moderate chalking and severe crazing during the second durability cycle. The water-borne acrylic sealant passed two durability cycles, when no fatigue cycling was included, but failed in the second durability cycle, when the specimens were exposed to fatigue cycling. This behaviour was observed, regardless whether the sealant surface was painted or not prior to weathering.

Based on the above findings, at the meeting of ISO TC59/SC8 in 2001, the Japanese delegation recommended the RTR method for development as an ISO standard. With the exception of the urethane-cure acrylic sealant, they considered the degradation of the various chemical types of sealants observed after exposure to the two durability cycles to be similar to the one observed in outdoor weathering. The study conducted by the Tokyo Institute of Technology will continue until completion of the third durability cycle. Further studies are planned by the Architectural Institute of Japan in order to investigate the correlation of the results observed in the RILEM accelerated durability test method with those obtained in outdoor exposure. It is assumed that adjustments in the balance between accelerated weathering, fatigue cycling and thermo-mechanical cycling will allow tailoring of the RTR test method to specific outdoor climate and exposure conditions.

## 5 CONCLUSIONS

Both studies found that the RILEM durability test method is able to differentiate between products with regard to their resistance to accelerated ageing and mechanical cycling. The type of failure and the changes in surface appearance observed during the test regime are similar to those observed in actual service conditions. Since different sealant products were evaluated in both studies, their results cannot be compared directly; however, it appears that the Japanese study, using a Xenon-light source, yields faster degradation than the British study, which employed a fluorescent light source. Fatigue cycling substantially accelerates sealant degradation, as found by both studies. Finally, the dynamic cure procedure can be used to simulate movement during cure. However, a discussion of the British test results within RILEM TC139-DBS suggests the results depend critically on the relation between the duration of the movement cycle (2.4 hours) and the speed of cure (e.g. pot-life) of the product.

## 6 ACKNOWLEDGMENTS

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# Design And Material Selection Factors That Influence The Service-Life And Utility Value Of Dual-Sealed Insulating Glass Units

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**Summary: Insulating glass units are exposed to a variety of environmental factors, such as temperature and atmospheric pressure fluctuations, wind loads, working loads, sunlight, water and water vapour. The service life of a sealed insulating glass unit (IGU) critically depends on the perfect functioning of the edge-seal under these environmental influences. The water vapour permeability of the secondary insulating glass sealant plays only a subordinate role in the life expectancy of a dual-sealed IGU, since the resistance of the edge-seal to water vapour diffusion is determined almost exclusively by the low water vapour permeability of the polyisobutylene primary seal. However, great importance must be attached to the viscoelastic properties of the secondary insulating glass sealants, particularly to their tensile stress behaviour and their elastic recovery under service conditions, as these properties affect the ability of the primary seal to function. The resistance of the edge-seal to gas diffusion is influenced by the permeabilities of both primary and secondary seals. IGUs with very low gas loss rates, meeting the stringent requirements of the German and new European standards, can be produced by proper selection and design of the edge-seal components, particularly by either minimising or properly accommodating the movements occurring in the edge-seal as a result of the thermal stresses. The durability of the secondary seal has a strong influence on the service-life of dual-sealed IGUs, as it affects the transport mechanisms for moisture ingress and gas loss.**

**Keywords: Insulating glass unit, Sealant, Durability, Service-Life, Life Expectancy.**

## 1 INTRODUCTION

The key function of an insulating glass unit (IGU) edge-seal system is to provide a gas- and moisture-barrier and to structurally bond two or more panes of glass together. This can be achieved best with a dual edge-seal system, where the primary, polyisobutylene-based seal provides the barrier function and the elastomeric secondary seal ensures the structural integrity of the IGU. In the year 2000, about 280 million $m^2$ of IGUs were manufactured globally. North America and Europe accounted for over 80% of this volume and about 2/3 of these IGUs were dual-sealed, with the market share of dual-sealed units being much higher in Europe (95%) than in North America (45-50%). Experience with dual-sealed IGUs has shown that service-lives of more than 25 years can be obtained, if the units are properly designed, manufactured and installed.

An IGU has reached the end of its service-life, when – under normal use conditions – moisture condensation (fogging) occurs within the inter-pane space. Fogging is caused by the diffusion of water vapour through the edge-seal and the resulting saturation of the desiccant. Modern, multi-functional IGUs are often filled with special gases, such as noble gases, $SF_6$ or mixture of these gases, in order to improve their heat and/or sound insulation quality. The differential in partial pressures of these gases within the IGU and the surrounding air causes them to diffuse, effectively resulting in a “dilution” of the gas(es) in the IGU by air. The dilution of the fill gases in turn affects the utility value of the IGU by degrading its heat and/or sound insulation quality. Thus, the quality of edge-seal strongly influences the service-life and utility value of an IGU.

## 2 ENVIRONMENTAL AND SERVICE DEGRADATION FACTORS

IGUs are exposed to various environmental and service factors that negatively affect their life expectancy (Wolf 1992, 1999). Literature on this subject often differentiates between physical and chemical stresses, although certain environmental influences, such as sunlight, exert both kinds of stress.

### 2.1 Temperature and its variation

High temperatures accelerate most physical and chemical processes, such as the ageing of the insulating glass sealant and the diffusion of gas or water vapour through the edge-seal. Temperature fluctuations produce variations in pressure within the

IGU. These exert mechanical stresses on the edge-seal, which are strongest for small units and unfavourable form factors (Feldmeier 1997). Furthermore, differences in thermal expansion of spacer bars and glass plates result in shearing and peeling forces in the edge-seal. The edge-seal temperature is influenced by the steady diurnal and annual ambient temperature variations; however, rapid changes in edge-seal temperature do occur; for instance, during a summer thunderstorm, when hail or cold water hits the heated outer glass pane. Even in quite moderate climates, such as Scotland, the edge-seal temperature in clear glass IGUs may vary by as much as 20°C within 2-3 hours (Garvin & Wilson 1998).

### 2.2 Air pressure and its variations

Permanent stresses on the edge-seal exert a highly negative influence on the service life of an IGU. Since they can be avoided through proper manufacture and installation procedures, this situation will not be further examined here. Changes in ambient atmospheric pressure result in temporary mechanical stresses on the edge-seal; these changes typically occur within the time span of several days. Much faster load changes occur with the sudden build up of pressure caused by a gust of wind or the change from pressure to suction produced with swirling winds. These rapid changes in wind load occur several hundred thousand times during the service-life of an IGU.

### 2.3 Sunlight

Sunlight exerts both physical and chemical stresses on the edge-seal. Visible and infrared components of the solar radiation thermally charge the IGU. The effect increases, if a tinted solar protection glass is involved, since then a larger portion of the visible light is also converted into thermal energy. The short wave component of the solar radiation may also induce photochemical processes in insulating glass sealants. Regular float glass is transparent to ultraviolet light down to a wavelength of about 280 nm (Spauszus 1975). In the case of a regularly glazed IGU, the unit's edge-seal is protected by the window frame; hence, no sunlight reaches the edge-seal directly. However, depending on the angle at which the sunlight strikes the glass surface, between 1% and 8% of the incident radiation finds its way to the edge-seal due to internal reflection in the glass panes (Van Santen 1984, 1986). The penetration length of this radiation into the depth of the rebate depends on the thickness of the glass pane (Marusch 1988). The glass adhesion of organic insulating glass sealants, such as polysulphides or polyurethanes, can be irreversibly destroyed by the high-energy short-wave spectrum (280-380 nm) of sunlight (Ludwig & Wolf 1986, Marusch 1988). Consequently, the edge-seal of organically sealed insulating glass panes is not to be freely exposed to the sunlight, but must be suitably protected, as in the case of sloped glazing, for example, by attaching strips, gaskets or tapes.

### 2.4 Water and water vapour

Water and water vapour may also cause both physical and chemical stresses on the edge-seal. Two of the major physical stresses are the water vapour diffusion through the edge-seal into the inter-pane space as well as the water absorption and the associated swelling of the insulating glass sealant itself. If the edge-seal of an insulating glass unit remains in direct contact with water for a protracted period of time, e.g. due to improper glazing, organic insulating glass sealants tend to absorb large quantities of water into their polymer matrix, thereby greatly increasing the volume of the edge-seal (Boesmans 1990, Ludwig & Wolf 1986). This swelling of the edge-seal inevitably results in an opening of the primary seal and, consequently, a higher rate of water vapour diffusion into the interior of the insulating glass unit. Glazing guidelines for IGUs therefore attach great importance to keeping the glazing rebate free of water. However, water and water vapour are also capable of triggering chemical reactions. By causing hydrolysis of chemical bonds at the glass surface, water can irreversibly damage the adhesion of insulating glass sealants (Ludwig & Wolf 1986, Shephard *et al*. 1999, Lowe 1992).

### 2.5 Synergetic effects

The stresses caused by the various environmental factors are not simply cumulative in their effect. Instead, their interaction results in a disproportionately higher stress on the edge-seal, a phenomenon known as synergism. Various studies (Feldmeier *et al*. 1984, Van Santen 1984, Ludwig & Wolf 1986, Lowe 1992) ascertained that the simultaneous action of water, elevated temperatures and sunlight constitutes the greatest stress on the edge-seal of an IGU. This degradative effect is most pronounced at the sealant/glass interface, often resulting in partial or complete adhesive failure of organic sealants, due to the channelling of UV radiation to the interface by total reflection within the glass panes. In the absence of water, free radicals are formed by the absorption of radiant energy and the resulting chain scission of organic polymers; however, these radicals tend to recombine, forming a somewhat different network configuration. In the presence of water, however, the radicals formed are terminated and cannot recombine, thus generating low molecular weight compounds that form a weak boundary layer. As a result, organic sealants exposed to UV light, water and elevated temperatures for prolonged periods of time fail progressively near the sealant/substrate interface.

## 3 DEGRADATION MECHANISMS AFFECTING INSULATING GLASS PERFORMANCE

Theoretical calculations based on moisture and gas diffusion through the edge-seal generally yield service lives far in excess of 25 years. Why is this theoretical life expectancy not achieved in normal service, with some units failing prematurely even within a few years of service? The theoretical calculations ignore the rigours that IGUs have to endure, the actions of sunlight, heat, wind loading, excessive moisture exposure due to improper glazing, etc., the resulting stresses, edge-seal defects due to poor workmanship, and – most importantly – the degradation of the edge-seal with ageing! The calculations are generally based on only one transport mechanism (diffusion through the edge-seal) and do not take account of changes in this transport mechanism or the advent of additional transport mechanisms resulting from the degradation of the edge-seal. The service life and gas retention of an IGU are influenced by the following factors:

- Exposure conditions (microclimate within window frame, external climate, wind loads, etc.)
- Dimensions of the insulating glass unit
- Quantity, type and initial loading of the desiccant
- Air or gas temperature and relative humidity during manufacture of the insulating glass
- Initial degree of gas filling
- Resistance of the edge-seal to diffusion of water vapour and gas
- Effective sealant cross-section through which the diffusion occurs
- Durability of the edge-seal

### 3.1 Diffusion through edge-seal

Considering all conditions as identical and neglecting the durability of the edge-seal for the time being, the service life and the utility value of an IGU should be the higher, the lower the amounts of water vapour and gas that diffuse through the effective cross-section of the edge-seal. The key parameters that need to be controlled, thus, are the permeability of the edge-seal and the effective cross-section.

#### 3.1.1 Permeability of the edge-seal

During service, the edge-seal of the glazed IGU is exposed to a microclimate within the window frame or curtain-wall construction. Two major studies have been conducted in an effort to monitor this microclimate in terms edge-seal temperature, moisture and presence of liquid water over the period of several years (Feldmeier *et al*. 1984, Garvin & Wilson 1998). While in Central Europe edge-seal temperatures of clear glass IGUs seldom exceed 40°C, for tinted or coated glass units or in warm climates service temperatures may well reach 80°C and above for prolonged time-periods (Jacob & D'Cruz 1999). As a rule of thumb, the glass temperature increases by 10°C for every 200 W/m$^2$ absorbed sunlight radiation. Since the diffusion of moisture or gas occurs under service conditions, the temperature dependency of diffusion through an insulating glass sealant must be taken into consideration, which can be described by an exponential function of the inverse, absolute temperature (Arrhenius equation), where $E_a$ is the activation energy of diffusion, R is the ideal gas constant, and T the absolute temperature:

$$\frac{P(T_2)}{P(T_1)} = \exp\left( -\frac{Ea}{R} \bullet \left( \frac{1}{T_2} - \frac{1}{T_1} \right) \right) \qquad (1)$$

Insulating glass sealants therefore exhibit a considerably higher moisture and gas permeability at elevated temperatures than at room temperature. For instance, as reported by Wolf (1985), the water vapour permeability of insulating glass sealants at 60°C is an average of six to eight times higher than at 20°C. The practical consequences of this must be examined. Although in Central Europe the edge-seal of an IGU is subjected to temperatures over 30°C for only about 20% of the year (Feldmeier *et al*. 1984), the IGU nevertheless sustains about twice as much damage by diffusing water during this brief summer period than during the remainder of the year, when temperatures under 30°C prevail. Higher temperatures in association with high humidity therefore drastically reduce the service life an IGU.

The activation energy of diffusion depends mainly on the polymer type utilised in the insulating glass sealant. The water vapour diffusion rates of silicone sealants vary less as a function of temperature than is the case for polyurethane or polysulphide sealants (Wolf 1985). As a result, the water vapour permeabilities of silicone and polysulphide insulating glass sealants tend to approach each other at elevated temperatures. Based on these findings, comparable service-lives should be expected for polysulphide and silicone single-seal IGUs – in contrast to frequently voiced arguments, which are based on water vapour permeabilities determined at room temperature. However, the service lives of such single-seal IGUs will be substantially shorter than those achieved with dual-sealed units.

Various authors have studied the temperature dependency of gas diffusion through organic elastomers, including PIB, silicone, and polyurethane elastomers (see Schuck 1980, Beckmann 1991, and literature cited therein). These authors consistently found silicone elastomers to have the lowest temperature dependency in gas permeability of all high molecular weight polymers studied. The activation energies for the diffusion of nitrogen and oxygen through silicone elastomers were 14 and 9 kJ/mole, respectively, while organic polymers typically showed activation energies for these gases in the range 28-52 kJ/mole. On the other hand, it should be noted that the silicone elastomers also had the highest absolute gas permeability of all elastomers studied.

However, since the majority of IGUs are dual-sealed, the diffusion resistance of a double layer system needs to be considered. The diffusion resistance of a plane-sheet laminate is the sum of the individual resistances of the various layers (Ashworth 1992):

$$\frac{d}{P} = \sum_{i=1}^{n} \frac{d_i}{P_i} \tag{2}$$

When considering moisture diffusion, the water vapour permeability of the polyisobutylene (PIB) primary seal is far lower than that of the secondary seal, irrespective of whether the secondary seal is made of a silicone, polysulphide or polyurethane sealant. Therefore, the water vapour permeability of the dual edge-seal is determined almost exclusively by the permeability of the PIB primary seal:

$$P_1 << P_2 \Rightarrow \frac{d}{P} \cong \frac{d_1}{P_1} \tag{3}$$

Experimental studies into dual edge-seal systems (Massoth & Wolf 1988) confirm this approximation; hence, it could be assumed that the service life of a dual-sealed IGU in the field would depend solely on the PIB primary seal, and that it is therefore irrelevant which sealant might be used for the secondary seal. This is not the case, however, as will be discussed later.

**Table 1. Overall argon gas permeabilities of dual edge-seals.**

| *Sealant Types* | *Argon Permeability* ($cm^2/(s\ cmHg)$) | |
|---|---|---|
| | Single Seal | Dual Seal |
| Polyisobutylene (PIB) | $5.0\ 10^{-11}$ | ---- |
| Polysulphide | $1.5\ 10^{-10}$ | $6.82\ 10^{-11}$ |
| Polyurethane (Polybutadien) | $8.0\ 10^{-10}$ | $8.00\ 10^{-11}$ |
| Polyurethane (Polyether) | $2.8\ 10^{-9}$ | $8.24\ 10^{-11}$ |
| Silicone | $3.7\ 10^{-8}$ | $8.33\ 10^{-11}$ |

In the case of gas diffusion, one needs to consider the individual contributions to the overall permeability of the dual-seal, which depend on the nature of the permeating gas. Table 1 shows the overall gas permeabilities of various dual edge-seal systems for argon, the most commonly used fill gas. The values were calculated from the experimentally determined gas permeabilities of the individual seals (Geilich 1987) using eqn. (2), and assuming primary and secondary seal depths of 6 and 4 mm, respectively. As can be seen, the overall gas permeabilities of polyurethane or silicone based dual seals are between 17% and 22% higher than the respective PIB/polysulfide dual seal. This is a rather small increase, when compared with the effect of the effective seal cross-section, through which the diffusion occurs (to be discussed later), or when considering the temperature dependency of gas permeability. For example, the nitrogen permeability of the polyurethane elastomer studied by Beckmann (1991) increased by a factor of 17, when changing the measurement temperature from 20°C to 80°C. This increase is typical for the organic elastomers studied, since the activation energy of 40 kJ/mole of the polyurethane elastomer corresponds quite well to the average activation energy found for this class of elastomers (43.9 kJ/mole). Since the ratio between the gas permeabilities of various gases in organic elastomers remains about the same (Schuck 1980), one can assume a similar increase for the argon permeability of the edge-seal. Given a defect-free primary seal installation, this suggests that the majority of IGU gas loss occurs during periods of elevated edge-seal temperature.

### 3.1.2 Effective diffusion cross-section

The first, and most important, factor that influences the effective cross-section for the diffusion of moisture and gases is proper workmanship during IGU manufacture. The primary seal must be properly dimensioned, must be free of voids, and completely wet the spacer and glass contact surfaces. For conventional IGUs (rigid spacer), the primary seal cross-section is determined by the pressing operation which occurs after the two glass panes and the PIB coated spacer have been assembled. The pressure must be maintained over a sufficient period of time to allow the PIB sealant to flow and to completely wet the spacer and glass surfaces. The strength and duration of the pressure must be controlled to achieve primary seal dimensions of 0.3-0.4 mm in width and about 5-6 mm in depth.

In the case of rigid spacers, the degree to which the primary seal opens up during periods of positive pressure differential is determined by the tensile stress with which the secondary seal resists this applied force. In practice, positive pressure differentials occur at low atmospheric pressures or at high temperature, whereby temperature is responsible for most pressure differentials. Temperature induced pressure differentials also excert a higher force on the edge-seal than do wind loads or atmospheric pressure variations (see Fig. 1, adapted from Van Santen 1986). Therefore, the tensile stress behaviour (Young’s modulus) of secondary sealants at elevated temperatures must be considered.

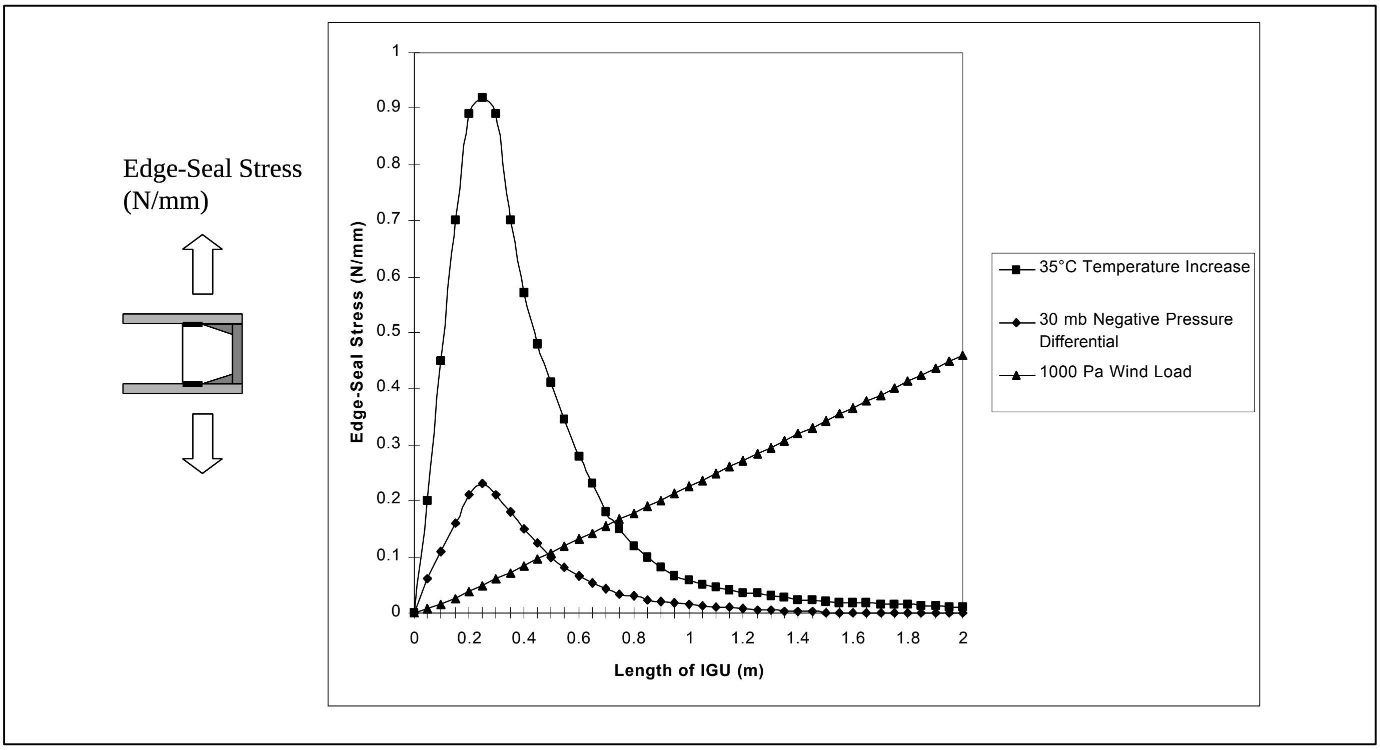


**Figure 1. Edge-seal stress as a function of IGU length for various loads**

O'Brien & Stewart (2000) modelled the thermal movements occurring in the edge-seal of a large IGU (1.5 x 2.1 $m^2$ size) as a result of temperature variations (-30°C to +60°C) for three commercially available spacer bars of different material and design using finite-element analysis. The model was based on nylon corner keys for the aluminium and galvanised steel spacers and bent corners for the stainless steel spacers. The nylon corner keys were assumed to be solid and firmly bonded to the spacers; while the bent corners were modelled as solid, bent metal corner keys, also firmly bonded to the spacers. Since actual bent corners are hollow, the model tends to overestimate the stresses for this corner design. As expected, at the low temperature, the corners are pulled inwards, resulting in a bending angle >90°; while at the high temperature, the corners are pushed outwards, resulting in a bending angle <90°. Figure 2 shows the deformed corner shapes for the aluminium spacers exaggerated by a factor of 100. Monitoring the changes occurring in PIB primary seal thickness around the circumference of the IGU, O'Brien & Stewart found that the stainless steel spacer had, by far, the least effect on the change in cross-sectional area, while the aluminium spacer had the largest effect, as can be seen in Table 2. This finding is in line with the sequence expected based on the difference in thermal expansion coefficients between spacer material and float glass. Thus, changes in the effective diffusion cross-section, resulting from a differential in thermal movements, are likely to account for the observation of performance differences of IGUs made with different spacer materials.

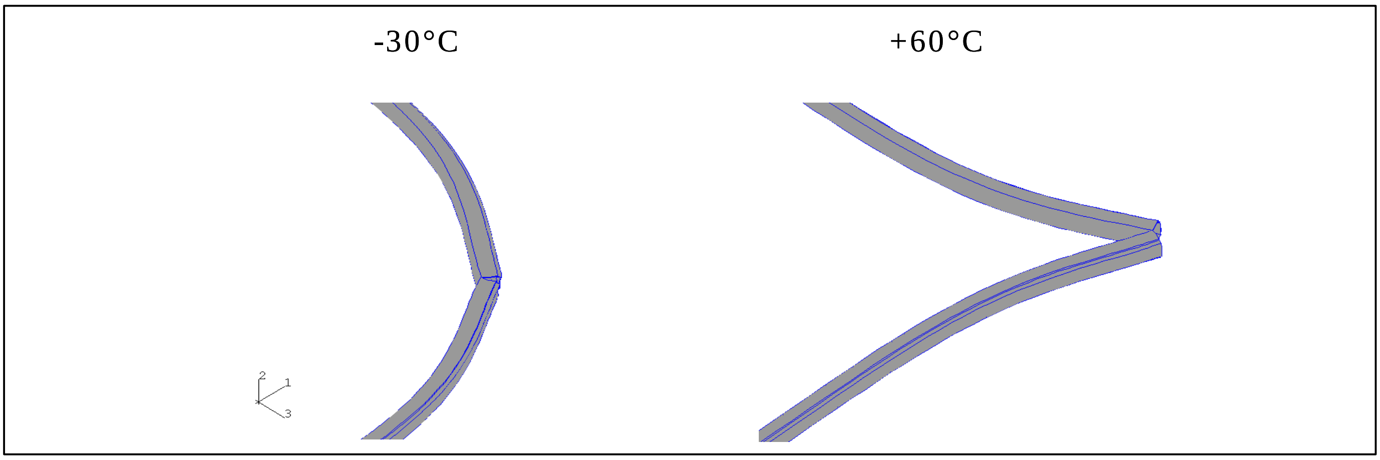


**Figure 2. Corner deflection of aluminium spacer frame (exaggerated by factor 100)**

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**Table 2. Deformation of PIB primary seal as a function of spacer material**

| *Temperature* | *Deformation of PIB seal (%) for spacer material* | | |
|---|---|---|---|
| (°C) | *Aluminium* | *Galvanised* | *Stainless steel* |
| +60°C | +51% | +42% | +6% |
| -30°C | -60% | -36% | -4% |

A further factor affecting the service life and utility value of an IGU is the time during which the opening in the primary seal occurs. Irrespective of the type of secondary sealant used, the opening occurs for the duration of the positive pressure difference. As discussed above, the degree of opening depends on the tensile stress of the secondary sealant. However, once the positive pressure differential decreases and an equilibrium is reached between inside and outside pressure, the length of time, required by the edge seal to close the primary seal, varies, depending on the degree of elastic recovery of secondary sealant utilised. Sealants with a low elastic recovery exhibit plastic flow in their stress strain behaviour; due to this relaxation mechanism their tensile stress decreases during maintained extension. As a result, when the applied force is eliminated, they are no longer capable of quickly closing the primary seal to its original size. If the secondary seal does not recover completely, the primary seal remains permanently deformed. Since the opening of the diffusion pathway occurs primarily at elevated temperatures, a secondary sealant with poor elastic recovery at elevated temperatures markedly shortens the service life of an IGU.

Apart from the fluctuations in temperature and atmospheric pressure, the ambient moisture level exerts an indirect influence on the opening and closing of the primary seal. At high moisture levels, and even more so in direct contact with liquid water, secondary sealants absorb water, which increases their volume and degrades their mechanical properties. In general, the lower the crosslinking density of the polymeric sealant network, the higher the water pick-up is. For the most part, the extent of the negative effects caused by the water pick-up is directly proportional to the amount of water absorbed. The swelling of the secondary sealant results in an opening of the primary seal, whereby the effective diffusion cross-section is increased.

### 3.2 Changes in transport mechanisms induced by edge-seal ageing

As mentioned before, exposure of an IGU to environmental or service degradation factors causes ageing of the edge-seal. This ageing can induce other transport mechanisms for the ingress of moisture or loss of gas. An indication that such ageing occurs during the service-life of an IGU is the increase in moisture penetration over time. Without ageing, under steady-state conditions, and at a time far from saturation of the desiccant, moisture ingress into an IGU per time unit should be a constant. As the IGU approaches saturation of the desiccant, moisture ingress should slow down, since the partial pressure differential equilibrates. However, measurements undertaken on units undergoing repeated humidity and temperature cycles in the laboratory as well as on those installed in the field often show a non-linear increase in moisture penetration with exposure time (Van Santen 1986, Marusch 1988), as shown schematically in Fig. 3.

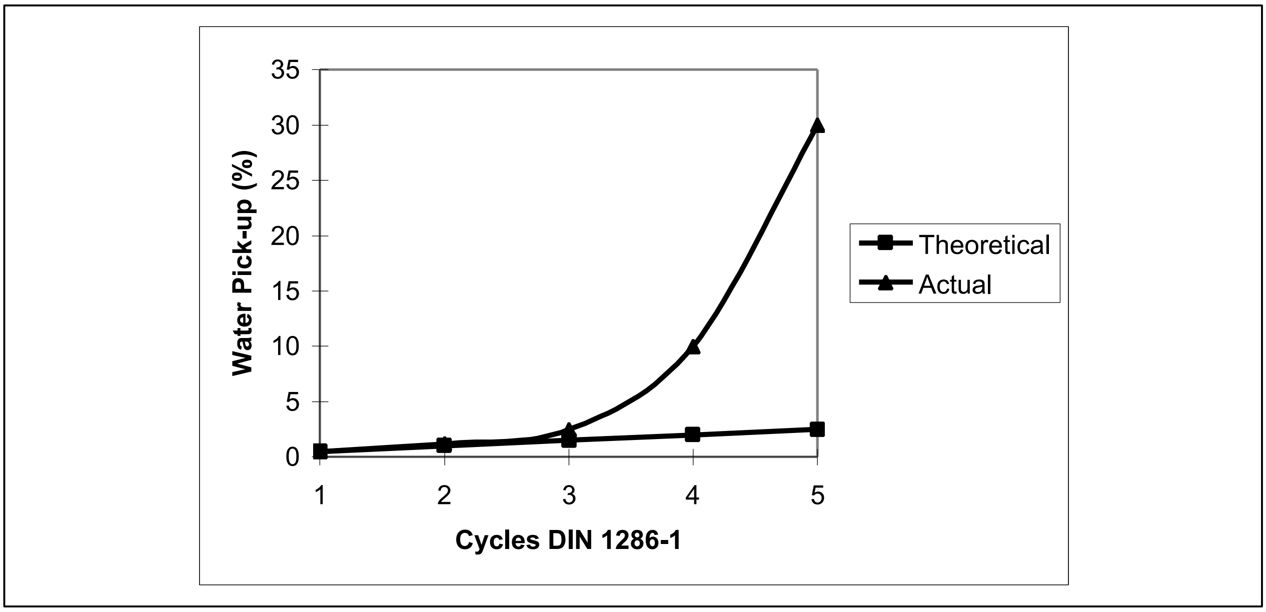


**Figure 3. Non-linear behaviour of actual water pick-up with IGU ageing**

One explanation for this phenomenon, favoured by Van Santen (1986), is the physical degradation of the primary seal, caused by the temperature and pressure induced movement in the edge-seal. Due to this repetitive movement, the primary seal may fail either cohesively or adhesively. Both failure mechanisms have indeed been observed in service, often combined with a migration of the PIB primary seal into the visible inter-pane space. Recently, much progress has been made in the design of spacer bars that allows reduction or elimination of this failure mechanism by restricting the ability of the PIB seal to migrate (Paci 1999).

As mentioned before, even in a regularly glazed IGU, a certain percentage of the incident light reaches the edge-seal via total reflection within the glass panes. When exposed to the destructive effects of the short wave spectrum of sunlight for prolonged periods of time, organic sealants may lose their glass adhesion. Schlensog (1986) monitored the evolution of organic sealant delamination from the glass substrate by using polarisation microscopy. The study showed that, before adhesive or boundary (thin-film) failure becomes visually detectable in the edge-seal, microscopic delaminations occur which, upon continued exposure, grow and interconnect, leading to macroscopic failure. It is highly likely that both moisture and gases migrate along this interfacial damage zone before macroscopic failure occurs, possibly with the moisture ingress accelerating this failure mechanism (Lefebvre *et al*. 1989); however, until now, this interfacial transport mechanism has not been experimentally verified for IGU edge-seals.

The repetitive shearing and peeling forces, caused by the differential in thermal expansion of spacer bar frame and glass plates as a result of edge-seal temperature variations, induce a high stress in the edge-seal, which may be compounded by ageing effects. Shear movements in joints induce a tensile stress component close to the sealant/substrate interface that is approximately twice as high as the original shear stress; this explains why sealants under shear tend to fail close to the substrate surface (Wolf & Iker 1992). If a secondary seal hardens in service, the resulting increase in tensile stress may cause partial or complete loss of adhesion.

## 4 PREDICTION OF IGU SERVICE-LIFE

Various national accelerated test methods have been developed for the assessment of IGU performance (Burgess 1999). All of these methods are acknowledged to be without scientific verification, yet the methods have some corroboration through years of field experience, and sometimes are backed by systematic field correlation studies. However, as discussions during the development of international ISO and EN standards have shown, there is no generally accepted understanding of the interactions of the degradative factors influencing the service-life of IGUs. As pointed out by Burgess, already early-on during the development of the Pr-EN 1279 standard (EN 2000), it became apparent that no one actually knew what test regimes were most onerous on IGU edge-seals. The investigations within the CEN committee showed that low temperatures did not cause IGU failure, but that sustained high humidity, rather than high humidity cycling, was the main driver of moisture into sealed IGUs. However, the committee felt that they could not ignore low temperatures or cycling and agreed to split the test regime into two parts, one with high humidity cycling, and one with constant high humidity and temperature. The CEN committee also agreed to assess the performance of IGUs based on the moisture pick-up of the desiccant, the method used in the German standard DIN 1286-1 (DIN 1994), since this method allowed establishment of relative performance levels. The CEN test method is now thought to be more onerous than any existing national standard and to ensure a service life of the IGU in excess of 25 years (Burgess 1999).

Table 3 shows the water pick-up measured on the desiccants of various dual-sealed IGUs after exposure to temperature/humidity cycling according to DIN 1286-1 (DIN 1994) as well as after three months of constant high temperature (55°C) and high relative humidity (100%) (Wolf 1992). As can be seen, substantially higher moisture penetration was observed in the constant climate test than in the cyclic test. Interestingly, silicone dual-sealed IGUs exhibited the lowest moisture penetration of all units tested; contrary to what would be expected when considering the water vapour permeability of the secondary IGU sealants alone. Under constant high temperature and humidity, the performance advantage of silicone sealed IGUs becomes even more pronounced; the moisture penetration of the organic sealed IGUs is about three times as high as that of the silicone sealed units. Since the desiccant's water pick-up is inversely proportional to the life expectancy of an insulating glass unit, this finding indicates the by far superior service-life of silicone dual sealed IGUs when compared to those sealed with polysulphide or polyurethane sealants.

**Table 3. Water penetration into IGU after different accelerated exposures.**

| *Secondary sealant* | *Water pick-up (weight %) after DIN 1286-1* | *Water pick-up (weight %) after 3 months 55°C and 100% rel. humidity* |
|---|---|---|
| Polysulphide 1 | 0.5 | 5.4 |
| Polysulphide 2 | 0.8 | 5.6 |
| Polyurethane 1 | 0.5 | 4.2 |
| Polyurethane 2 | 0.6 | 4.5 |
| Silicone 1 | 0.4 | 1.5 |
| Silicone 2 | 0.4 | 1.6 |

The CEN committee also decided to adopt the requirements of the German DIN 1286-2 standard (DIN 1989) by restricting the gas loss rate to <1.0% per year. Annex B of pr-EN 1279-3 (EN 2000b) discusses the relationship between artificial and natural ageing with regard to thermal and sound insulation. Based on studies done on gas losses experienced by IGUs over a period of 10 years of service, the CEN committee assumes that a gas loss rate <1.0% per year after accelerated ageing ensures a total gas loss of <5% over 25 years of service. On the assumption that the improvement of the U-value with 100% argon filling is 0.4 $W/(m^2K)$, this gas loss results in a deterioration of $\Delta U < 0.04\ W/(m^2K)$. Gas-filled IGU passing the requirements of DIN 1286-2 or Pr-EN 1279-3 are being manufactured successfully with polysulphide, polyurethane or silicone secondary sealants. However, since the requirements for gas-filled IGUs are especially demanding, some design and material selection factors must be carefully considered.

## 5 INFLUENCE OF DESIGN AND MATERIAL SELECTION ON SERVICE-LIFE

Table 4 provides an overview of the key material-specific performance characteristics of elastomeric secondary IGU sealants (for a more detailed discussion see Wolf 1992).

**Table 4. Performance characteristics of elastomeric secondary IGU sealants.**

| *Material property* | *Polysulphide* | *Polyurethane* | *Silicone (alkoxy)* |
|---|---|---|---|
| Resistance of glass adhesion to sunlight | Poor | Poor | Excellent |
| Resistance of adhesion to water (long-term exposure) | Good to moderate | Moderate | Excellent to moderate |
| Elastic recovery | | | |
| at 23°C | Moderate | Good | Excellent |
| at 60°C | Poor | Moderate | Excellent |
| Change in Young's modulus with temperature | Very high | Moderate | Low |
| Water-swelling | Very high | High | Low |
| Water-vapour permeability | | | |
| (3 mm sheets) $(g/(m^2d)$ | 7-9 | 3-6 | 7-16 |
| at 20°C | 40-60 | 20-30 | 40-100 |
| at 60°C | | | |
| DIN 1286-1 water pick-up (weight %) | 0.5-1.2 | 0.4-0.7 | 0.4-0.6 |
| dual-sealed IGU | | | |
| Argon permeability (0.6 mm sheets) | | | |
| $(10^{-10}\ cm^2/(s\ cmHg))$ | 1.5-1.8 | 8-30 | 250-400 |
| DIN 1286-2 gas loss (% p.a.) | 0.4-0.9 | 0.6-1.0 | 0.7-1.0 |
| dual-sealed IGU | | | |

Clearly, the key performance advantage of polysulphide sealants is their low gas permeability, which allows them to tolerate poor workmanship, at least up to a certain degree. The poor resistance of glass adhesion to sunlight prohibits the use of polysulphide and polyurethane sealants in structural glazing or roof glazing applications. Polyurethane generally show better physical properties and lower water-vapour permeability than polysulphide sealants; their main selling feature, however, is their lower price. The past ten years have seen an overheated competitition between polysulphide and polyurethane IGU sealants for market share in Europe. The resulting price war has forced some manufacturers to drastically lower the polymer content of their sealants and finally has caused the major manufacturer of polysulphide polymer and sealants to withdraw from the market (Anonymous 2001). There are now indications that premature failure rates of some polysulphide sealed units in the field have increased again, at least for certain installation conditions (wooden window frames) in Germany, ending the period of more than 20 years of increasing IGU performance achieved by continuous development of IGU components, assembly and quality assurance methods.

Silicone sealants, on the other hand, excel in the resistance of their glass adhesion to sunlight, making them the material of choice for structural and commercial glazing as well as demanding roof glazing applications. With over 20 years of experience with silicone sealants globally, the excellent performance and service-life of silicone dual-sealed IGUs have been demonstrated. One drawback of silicone sealants is their high gas permeability. However, recent experience has demonstrated that argon-filled, silicone dual-sealed IGU can be manufactured, which reliably pass DIN 1286-2 requirements, if certain design, manufacturing and quality assurance aspects are being met.

First, it has been demonstrated that IGU edge-seal systems, which have the ability to accommodate some movement within the spacer itself, place less stress on the primary seal and thus ensure low gas loss rates under accelerated and actual service-life conditions. Examples of such spacers that have been used in IGUs passing DIN 1286-2 are the Chemetall or Teroson TPS (see, for instance, Unger 1999) and Edgetech Super Spacer®. Second, IGU edge-seal systems that minimise differential thermal movement, especially within the sensitive corner region, tend to perform significantly better in terms of gas loss rates than systems with high thermal movements, as discussed earlier. IGUs of different designs, but based on stainless steel spacers, have passed the DIN 1286-2 requirements, while same or similar systems based on aluminium or galvanised steel spacers have failed. Companies that have successfully qualified IGUs based on stainless steel based edge-seal designs are St. Gobain, Cardinal Glass, Veltherm, and Interpane. Third, when using stainless steel spacers, those designs that minimise the amount of PIB primary seal migration into the inter-pane space should be preferred.

Bent spacer-frame corners, gas-filling techniques integrated into the IGU assembling process (rather than filling via holes drilled into the spacer), improved, semi-automatic PIB application equipment and in-line (heated) PIB primary seal presses have all substantially contributed to the minimisation of gas loss and helped with the improvement of quality and service-life of IGUs. The trend towards “warm-edge” IGUs favours the above mentioned edge-seal systems. Today, silicone dual-sealed IGUs can be produced that not only excel in their durability and longevity, but also reliably meet the stringent requirements for gas retention, and therefore provide optimum service-life and utility value.

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# The Influence Of Leaching Of Contaminants On Strength And Durability Of Contaminated Recycled Brick Aggregate Concrete

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**Summary:The engineering and durability properties of concrete produced using recycled concrete aggregate have been thoroughly investigated in the UK and elsewhere. However little research data is available on the effects of using Contaminated Recycled Brick Aggregate (CRBA) on concrete strength and durability, or the likelihood of the contaminants leaching from concrete. The objective of the work presented here was to examine the influence of using CRBA on compressive strength and workability, to investigate leaching of contaminants from concrete and to assess the effect of leaching on the strength and durability properties of concrete.**

**Known concentrations of heavy metals (Cr, Zn, Ni, Pb, Cu) were dissolved in the mix water before it was added to the clean crushed brick aggregate and cement to produce concrete cubes. The likelihood of the heavy metals leaching from the concrete was investigated using a monolithic leaching test. The results show that the contaminants were encapsulated in the concrete; with release several orders of magnitude lower than the concentrations originally present. A comparison between concrete subjected to leaching and control specimens that had not undergone the leaching test show that the effect on compressive strength was not significant, however the water permeability of concrete was found to increase substantially as a result of leaching. DTA/TG has been used to explain this increase in water permeability, which was found to be due to the leaching of calcium hydroxide.**

**Keywords: Recycled Brick Aggregate; Toxic metals leaching; Compressive strength; Water permeability**

## 1 INTRODUCTION

A current challenge for the construction and aggregate industry is the need to move towards a more sustainable approach to the production of aggregate and the financial and environmental pressure to reduce the volume of waste going to landfill sites. The production of concrete and masonry waste in England and Wales in 1999 was estimated at 33.8Mt (Symonds, 2001). Since the introduction of the landfill tax in the UK, and with the declared introduction of an aggregates tax for primary materials, pressure to increase the use of recycled demolition wastes has intensified. The utilisation of *clean* coarse recycled concrete aggregate has been investigated in some detail and there is considerable guidance on its use within UK specifications (Collins & Sherwood 1995; Collins 1998; Dhir 1998). However the same level of detailed information does not exist for recycled brick aggregate concrete. Furthermore, the re-development of brown-field locations and the demolition of buildings on these former heavy-industry sites often generate a large quantity of demolished masonry which is contaminated with the by-products of industrial processes. As well as avoiding the rising costs associated with primary materials, the re-use of contaminated masonry in concrete avoids the expense of transportation and disposal, which is often prohibitive in the re-development of brown-field sites.

Previous published work has shown that it is possible to produce concrete of an acceptable standard using clean crushed brick aggregate (Hansen 1992; DeVenny & Khalaf 1998). However, information on the effect of using Contaminated Recycled Brick Aggregate (CRBA) on engineering and durability properties of concrete has to date not been widely available. Concerns over toxic trace metals in concrete produced with CRBA coming into contact with ground waters means that the environmental compatibility of the concrete is a prime concern. If concrete is designed to have very low permeability and connected porosity, the amount of trace metals leaching out of the concrete will be very small. Nevertheless, elevated levels of trace metals in concrete could pose a significant contamination threat to the environment and the leaching behaviour must therefore be assessed before it can be used with confidence by the industry. There is a great deal of literature regarding the potential of cement in combination with wastes to prevent the leaching of heavy metals into the environment. Glasser (1996) notes some of the key physical and chemical features of cement systems that may also be applied to CRBA concrete. Effective encapsulation of toxic metals and satisfactory service life demands concrete with low permeability, strength, integrity, long term durability and maintenance of a chemical potential for immobilisation. This paper aims to study the influence of using CBRA on workability and strength and the effect of leaching on the strength, permeability and microstructure of CRBA concretes.

## 2 EXPERIMENTAL PROGRAMME

Concrete mixes were prepared using contaminated crushed brick with Ordinary Portland Cement (OPC) at different water cement ratios. The workability of the concrete was measured using the slump test and compressive strength was determined at 7, 28, 56 and 90 days. Specimens were subjected to a monolithic leaching test with subsequent testing of strength and permeability to determine the effects of leaching on these properties. Samples were then analysed using DTA/TG to investigate the influence of leaching on the microstructure of the concrete.

### 2.1 Materials and sample preparation

The coarse aggregate used was crushed waste clay bricks with a maximum size of 20mm, crushed using a jaw crusher. A typical particle size distribution for CRBA is presented in Table 1. The physical properties of the aggregate was assessed at regular intervals and carefully controlled throughout the experimental programme. Table 2 outlines the physical properties of the RBA used for the concrete production. The fine aggregate used was medium sand (BS 812:1001). Both the coarse and fine aggregates were used in the saturated surface dry condition. Portland Cement conforming to BS 12 was used as the cementitous material.

**Table 1. Typical particle size distribution of CRBA**

| *Sieve Size (mm)* | *Mass retained(g)* | *Percentage retained* | *Cumulative % passing* | *Cumulative % retained* |
|---|---|---|---|---|
| 20 | 30.0 | 1.00 | 99.00 | 1.00 |
| 14 | 1061.6 | 35.39 | 63.61 | 36.39 |
| 10 | 920.1 | 30.67 | 32.94 | 67.06 |
| 6.3 | 678.3 | 22.61 | 10.33 | 89.67 |
| 5 | 190.0 | 6.33 | 4.00 | 96.00 |
| <5 | 120.0 | 4.0 | - | - |
| Total = 3000 | | Fineness modulus = 2.9 | | |

Aqueous solutions containing the heavy metals Copper (Cu), Chromium (Cr), Lead (Pb), Nickel (Ni) and Zinc (Zn) were added to the coarse aggregate prior to concrete production. The solutions were individually prepared by dissolving salts of $Cu^{2+}$, $Pb^{2+}$, $Ni^{2+}$, $Zn^{2+}$ (positive ions as nitrate) and $Cr^{6+}$ (as a negative ion in potassium dichromate) in deionised water. The levels of heavy metal contaminants added per unit mass of the crushed brick aggregate are given in Table 3.

**Table 2. Physical Properties of CRBA**

| | |
|---|---|
| Apparent specific gravity | 2.64 |
| Water absorption | 6.33% |
| Bulk Density (compacted, dry) | 1186 kg/m$^3$ |
| Bulk Density (uncompacted, dry) | 1068 kg/m$^3$ |
| 10% Fines Value | 101.5kN |
| Flakiness Index | 38.5 |

Guideline contamination values are also given, where the concentrations shown are considered to be associated with heavily contaminated material. As it can be seen from Table 3, the quantity of the contaminants in the RBA was exactly double suggested in the Dutch specifications (MHSPE 1998) for intervention.

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**Table 3. Typical guideline concentrations for the heavy metals investigated and level of contamination associated with CRBA used as coarse aggregate in the experimental programme**

| *Contamination guidelines and contaminant levels associated with CRBA* | *All values in mg/kg* | | | | |
|---|---|---|---|---|---|
| | *Cr* | *Ni* | *Cu* | *Zn* | *Pb* |
| ICRCL 59/83 contamination action value (ICRCL 1987) | 600 | 70 | 130 | 300 | 500 |
| Dutch contamination intervention value (MHSPE 1998) | 380 | 210 | 190 | 720 | 530 |
| Quantity added per unit mass of aggregate | 760 | 420 | 380 | 1440 | 1060 |

The mix proportions of the concrete used in the study are given in Table 4. The mix proportions were determined to produce concrete with different permeation properties and were adjusted using the yield equation.

**Table 4. Mix Proportions**

| *Mix No.* | *Cement Content kg/m$^3$* | *W/C Ratio* | *A/C Ratio* | *Target Compressive Strength* |
|---|---|---|---|---|
| 1 | 420 | 0.5 | 4.12 | 50 N/mm$^2$ |
| 2 | 350 | 0.6 | 5.14 | 40 N/mm$^2$ |
| 3 | 300 | 0.7 | 6.16 | 30 N/mm$^2$ |

The specimens used throughout the investigation were 100mm cubes. The cubes were cast de-moulded after 24hrs, sealed in a damp proof membrane and stored at 20$^0$C. The reason for this curing regime was to prevent exposure to the atmosphere whilst avoiding any leaching of water soluble heavy metals which would occur if the samples were cured using the conventional water bath method.

**2.2 Leaching test and leachate analysis**The leaching tests were conducted in triplicate on specimens that had been cured for ~28 days. The leaching test used was the Dutch NEN 7345 tank test method (NNI 1995). In the test, the leaching of components from the concrete was determined by submersing a test piece in 5 litres distilled water adjusting to pH 4, and renewing the leachant at 8 fixed intervals: 0.25, 1, 2.25, 4, 9, 16, 36 and 64 days. The collected leachate was acidified with nitric acid to pH<2 to prevent loss of metal ions prior to the analysis. The concentration of heavy metals was determined using ICP-MS and presented as cumulative leaching per unit mass of concrete (mg/kg) as a function of time. ICP-MS analyses were performed on a Fisons Instrument VG Plasma-Quad. Calibration standards were prepared using standard solutions containing known concentrations of chromium, nickel, copper, lead and zinc, also acidified to pH<2. These standards were used to check the quantitative determinations throughout the leaching investigation giving consistency to the results obtained.

### 2.3 Water permeability, strength and DTA/TG testing after leaching

After the leaching tests had finished, the test pieces were exposed to 20$^o$C, RH 55%, along with control samples that had not undergone the leaching test. The compressive strength and water permeability of these specimens was determined and profile grinding of selected specimens was undertaken to generate samples for DTA/TG analysis. The water permeability was measured using the "Autoclam" Permeability System, an instrument developed at Queen's University for comparison of permeability indices between different concrete mixes. DTA/TG analysis was done on a Netzsch Simultaneous Thermal Analyser STA 449C Jupiter machine, in the temperature range 0-1000$^o$C with a heating rate of 10$^o$C/min. Approximately 70mg sample was analysed under a nitrogen environment, with the same quantity of alumina used as a reference material.

## 3 RESULTS AND DISCUSSION

The results obtained are grouped into three categories, viz. control properties, leaching characterisation and effect of leaching. Control properties are those properties of both fresh and hardened concrete measured prior to carrying out the leaching tests.

### 3.1 Control properties

Table 5 reports the slump and the compressive strength of concrete manufactured with the CRBA aggregate. In Fig. 1, the strength development of the CRBA concrete is presented.

**Table 5. Slump and Compressive Strength of CRBA Concrete Mixes**

| *Mix No.* | *W/C Ratio* | *Slump (mm)* | *Target Comp. Strength($N/mm^2$)* | *Measured 28 day Strength ($N/mm^2$)* |
|---|---|---|---|---|
| 1 | 0.5 | 18 | 50 | 57.8 |
| 2 | 0.6 | 22 | 40 | 49.3 |
| 3 | 0.7 | 30 | 30 | 40.6 |

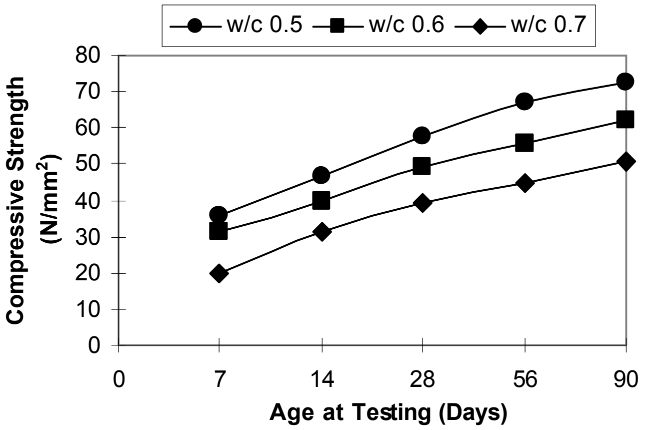


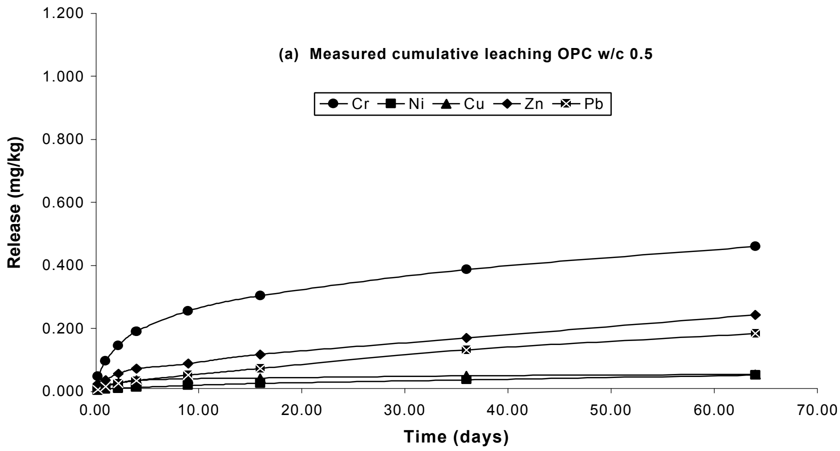

**Figure 1. Strength Development of CRBA Concretes**

From Table 5 it can be seen that, although the slump increased due to the increase in W/C ratio, all the mixes resulted in rather low slump values. In the context of mix design, all the slump values were found to fall in the range 10-30mm (DoE 1988). The workability could be improved by either increasing the water content in the mixes or adding water-reducing admixtures. The addition of further water could result in segregation and a significant decrease in compressive strength.

The compressive strength of the CRBA concretes reached their target values within 28 days (Table 5). In Fig. 1, the compressive strength was found to increase beyond 28 days, reaching significantly higher values for all the mixes at the age of 90 days. Better interlocking of crushed brick was thought to contribute to the strength. The results in Fig. 1 would suggest that the CRBA concretes could be used in structural applications.

### 3.2 Leaching characterisation

Figure 2 shows the cumulative leaching of the contaminants from the concrete cubes at different test intervals and Table 6 summarises the cumulative leaching at the end of 64 days of the leaching test.



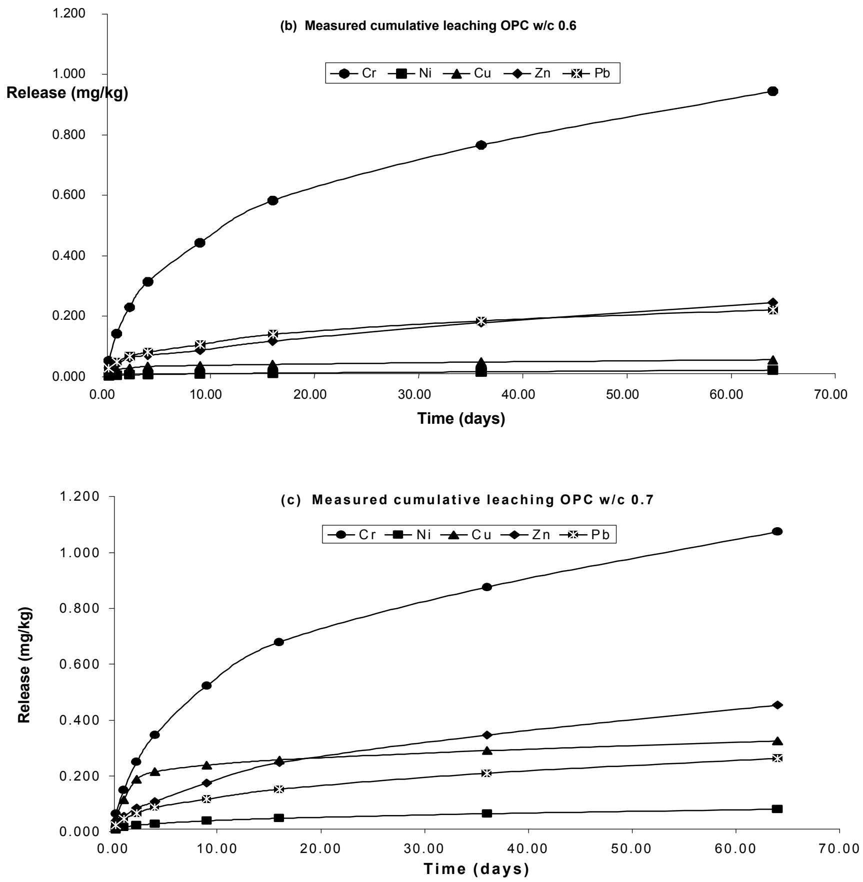



**Figure 2. Cumulative leaching of contaminants at different W/C ratios**

**Table 6. Cumulative leaching after 64-days leaching test and comparative data**

| | | *Cr* | *Ni* | *Cu* | *Zn* | *Pb* |
|---|---|---|---|---|---|---|
| Directive 98/83/EC - drinking water quality (μg/l) | | 50 | 20 | 2 | - | 10 |
| Cumulative leaching after 64days (μg/l) | 0.5 W/C | 198 | 22 | 23 | 102 | 79 |
| | 0.6 W/C | 403 | 9.5 | 24 | 105 | 94 |
| | 0.7 W/C | 451 | 34 | 136 | 190 | 110 |
| Quantity added per unit mass of concrete (mg/kg) | | 330 | 185 | 165 | 630 | 465 |
| Cumulative leaching after 64days per unit mass of concrete (mg/kg) | 0.5 W/C | 0.460 | 0.052 | 0.054 | 0.238 | 0.184 |
| | 0.6 W/C | 0.944 | 0.022 | 0.056 | 0.246 | 0.221 |
| | 0.7 W/C | 1.074 | 0.081 | 0.325 | 0.454 | 0.263 |

In Table 6, the amount of the contaminants permitted in drinking water in accordance with Directive 98/83/EC (EC 1998) is also presented. However, it must be noted that the water quality directive uses a spot sample of water intended for human consumption (e.g. tap water), which differs significantly from the leachate from the accelerated leaching test. Therefore, a direct comparison of the two sets of data is not appropriate. The cumulative leaching after 64 days in μg/l in Table 6 is presented to indicate that, whereas nickel quantities were not significantly different from those values in the Directive, the amount of copper, chromium and lead exceeded by a significant factor. The quantity of chromium and lead used to spike the aggregate was much higher than that used for both nickel and copper. Therefore, there could be preferential diffusion of the former contaminants.

In Table 6, the quantity of contaminants added to the aggregate per unit mass of concrete is also reported. While comparing the cumulative leaching of the contaminants with the amount that was used to contaminate the aggregate, the leaching could be considered to be negligible. In Fig. 2, the cumulative leaching varied with the water-cement ratio, as one would expect. The largest amount of leaching was found to be at the highest water-cement ratio, with a decrease in the leaching of all the

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contaminants with the reduction in the water-cement ratio. In all water-cement ratios, the largest leaching was observed for chromium, with no distinct trends between the other contaminants at all the three water-cement ratios. However, there were two distinct bands below chromium at both 0.5 and 0.6 water-cement ratios. Both zinc and lead followed chromium at these two water-cement ratios and the lowest cumulative leaching was observed for both nickel and copper. In the case of 0.7 w/c, these trends were not repeated. However, a close examination would reveal that copper leached out at a substantial quantity initially during the test, but later on it was similar to that obtained with nickel. Therefore, it can be said that both nickel and copper leached out in smaller quantities at this water-cement ratio as well. A comparison of the cumulative leaching at 64-days with those used to spike the RBA, would suggest that the amount of contaminants leaching is related to the amount of ion availability in the RBA. Fig. 2 shows also that the amount of contaminants leached out of the CRBA concretes continued even after 64 days. Therefore, it would be necessary to predict the rate of leaching from the calculation of the diffusion coefficient for the contaminants in order to predict the likely total amount that would leach out after a certain period of exposure. This was considered to be beyond the scope of this paper.

### 3.3 Effect of leaching on properties of concrete

The amount of calcium hydroxide in the sample is used in order to discuss the effect of leaching on the microstructure of the concretes. This was obtained from the DTA/TG graphs by observing the exothermic changes at about 460 degree Celsius. Figure 3 shows the DTA/TG results for 0.6 W/C CRBA concrete at two depths from the surface, which is the specimen not subjected to the leaching test. Figures 4 and 5 show respectively the effect of using distilled water and acidified water as the solution used in the tank leaching test.

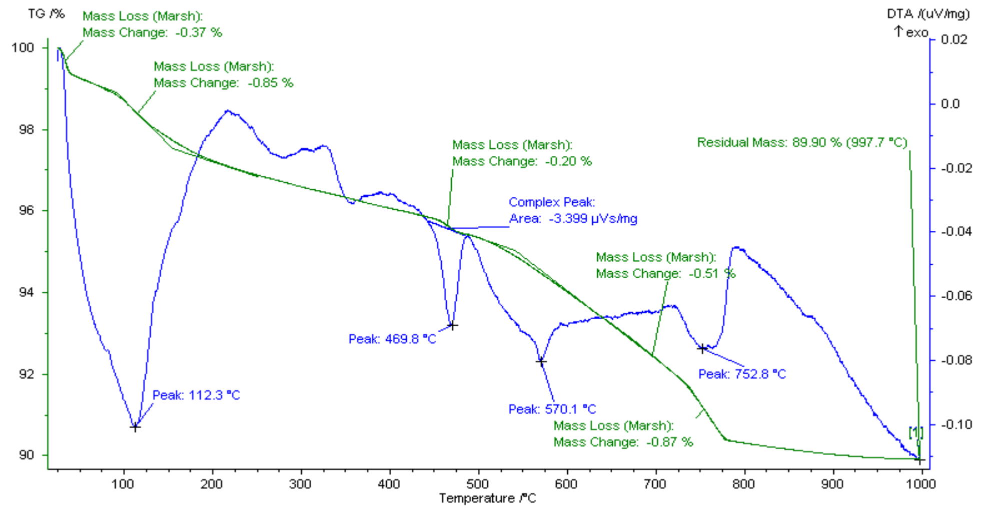


**Figure 3a. DTA/TG plot for 0.6 W/C concrete with no leaching (0-1 mm)**

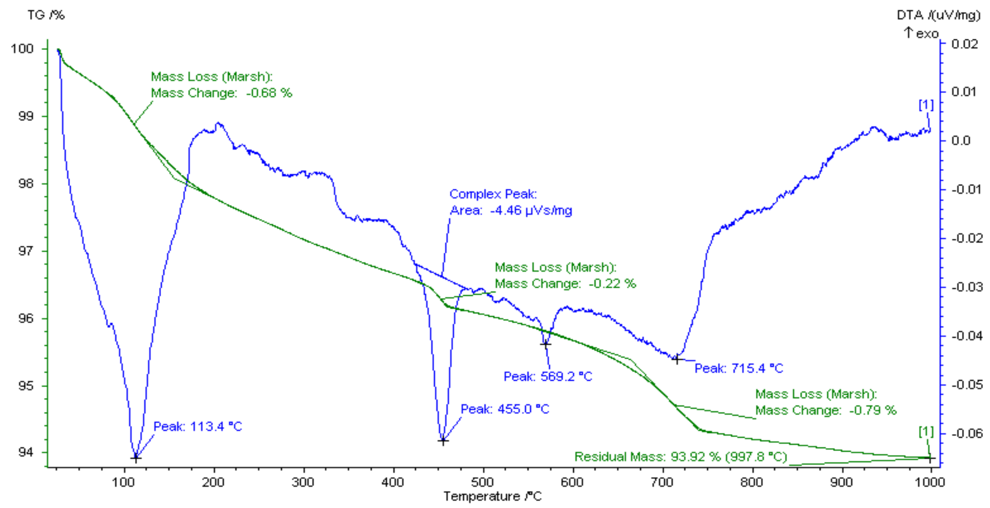


**Figure 3b. DTA/TG plot for 0.6 W/C concrete with no leaching (1-2 mm)**

The relative proportion of calcium hydroxide in Fig. 3a is 20% and that in Fig. 3b is 22%. These data would suggest that there was slightly better degree of hydration at the second depth; however, overall the results are very close. However, when both distilled water and acidified water were used to carry out the test, there was no peak corresponding to the presence of calcium hydroxide at 0-1mm depth (Figs. 4a and 5a) and only insignificant peak at 1-2mm depth (Figs. 4b and 5b). Between the distilled water and the acidified water, the latter was found to be more aggressive in removing the calcium hydroxide, as can be seen from the reduction in the peak corresponding to calcium hydroxide. Other peaks in the DTA/TG graphs are not discussed in this paper, primarily because the minerals in RBA were considered to be affecting the presence of different peaks rather than the decomposition of the cement hydrates.

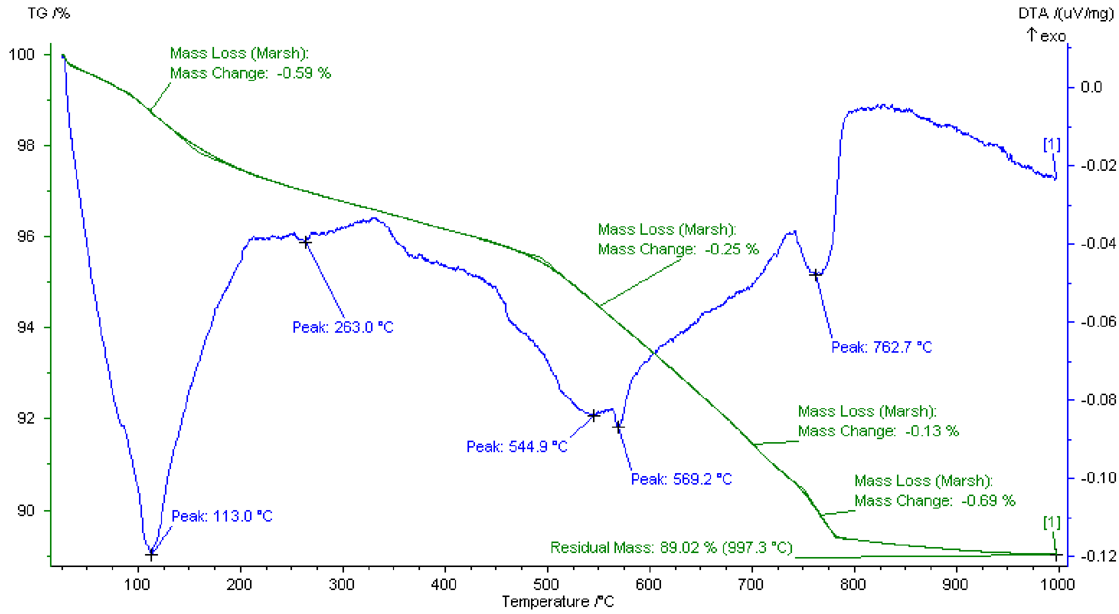


**Figure 4a. DTA/TG plot for 0.6 W/C concrete leached with distilled water (0-1 mm)**

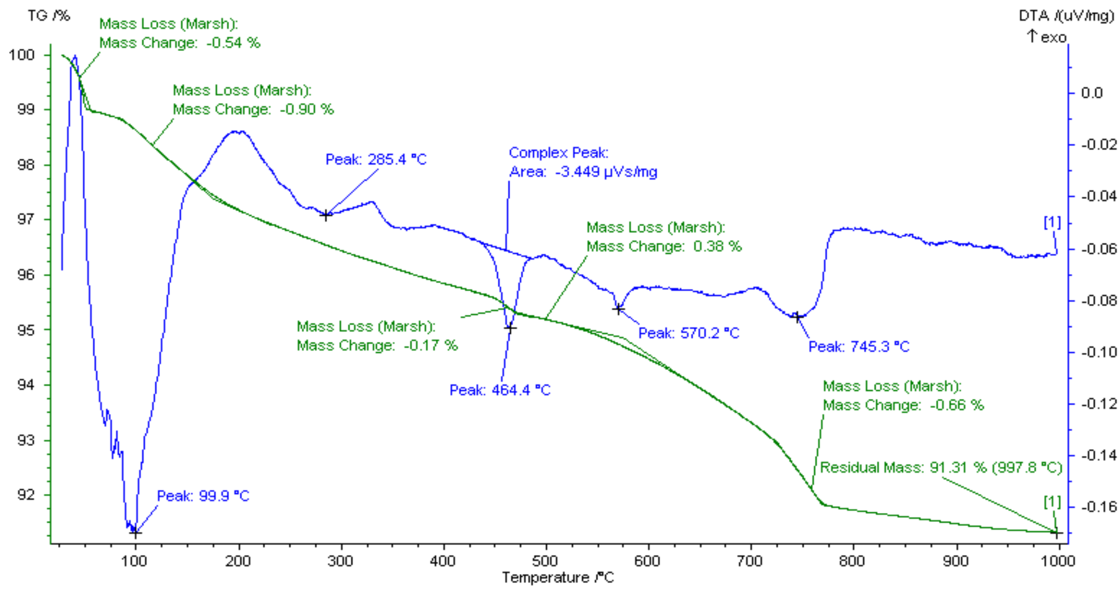


**Figure 4b. DTA/TG plot for 0.6 W/C concrete leached with distilled water (1-2 mm)**

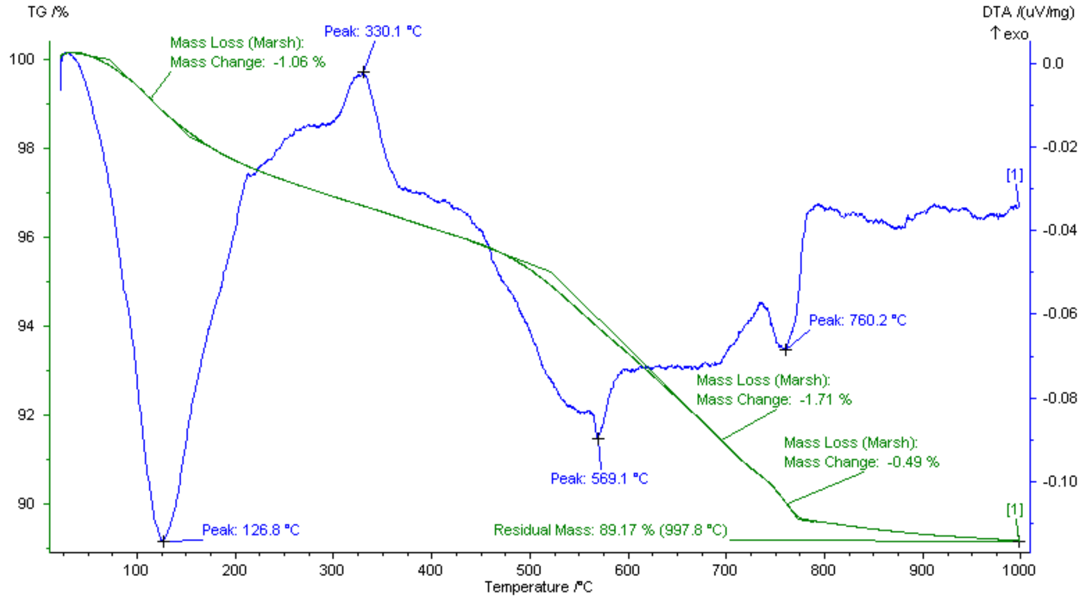


**Figure 5a. DTA/TG plot for 0.6 W/C concrete leached with acidified water (0-1 mm)**

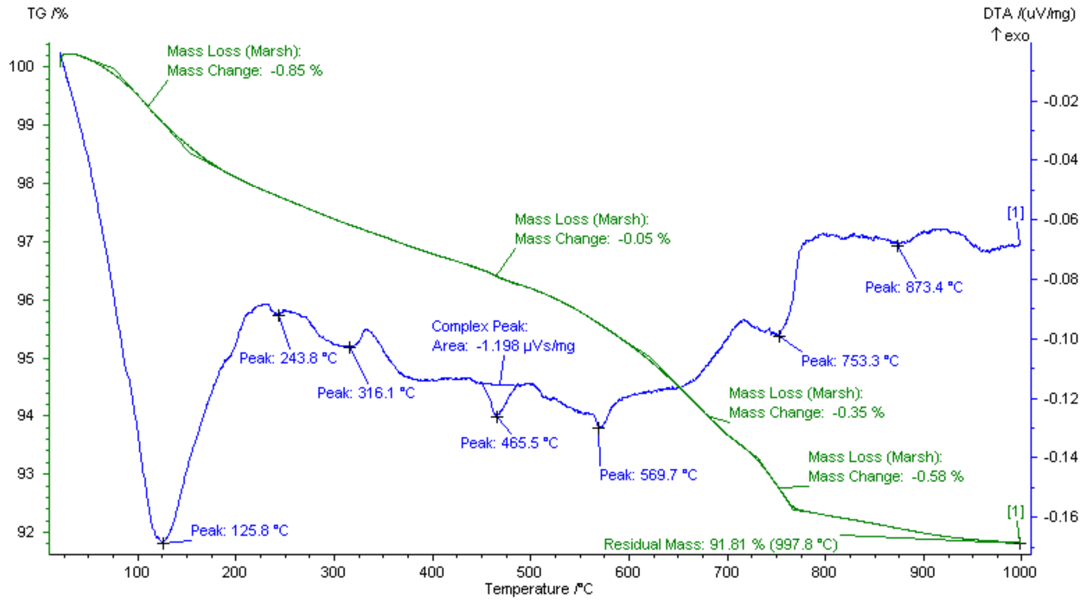


**Figure 5b. DTA/TG plot for 0.6 W/C concrete leached with acidified water (1-2 mm)**

The effect of the leaching on both strength and water permeability is presented in Figs. 6 and 7 respectively. In Fig. 6, the strength was not affected to any significant degree by the leaching by both distilled water and the acidified water. This is considered to be due to the fact that the microstructure of the solid products of the hydrated cement paste was presumably not affected by the leaching. The DTA/TG plots confirm that the peaks corresponding to calcium silicate hydrates were not affected by the leaching. However, the removal of the calcium hydroxide did change the water permeability of the concrete. At all the three water-cement ratios, the distilled water and the acidified water resulted in an increase in water permeability, in this order, when compared to control samples, which were not subjected to any leaching. Therefore, it is likely that the long-term durability of the CRBA concretes could be detrimentally affected by the leaching, even if the strength was found to be satisfactory.

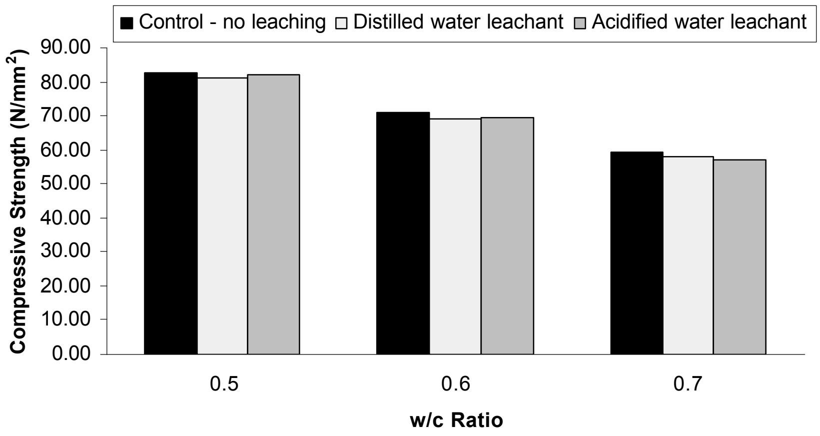


**Figure 6. Effect of leaching by different solutions on compressive strength**

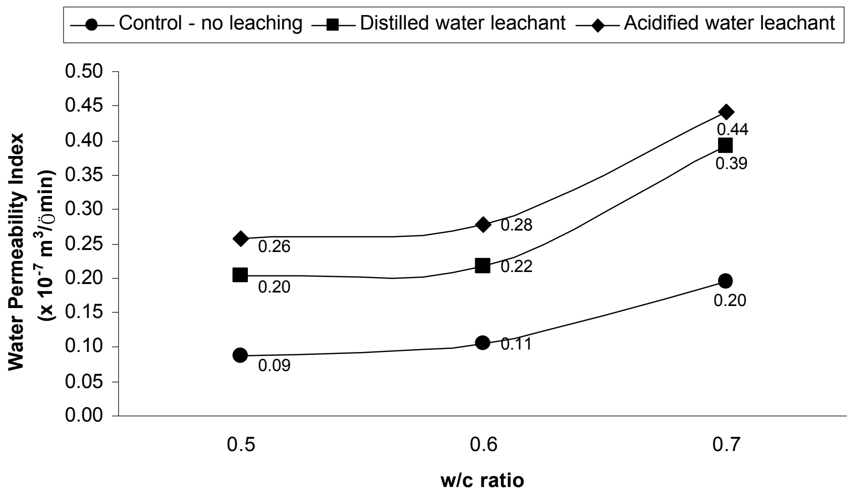


**Figure 7. Effect of leaching by different solutions on water permeability**

## 4 CONCLUSIONS

On the basis of the experimental programme presented in this paper and the results obtained, the following conclusions have been drawn:

(1) The contaminated recycled brick aggregate could be used to produce concretes which would satisfy the criteria of compressive strength. This would mean that the CRBA concretes could be used in structural applications.

(2) The leaching of contaminants was found to be insignificant while comparing with the amounts used to contaminate the aggregates. However, the cumulative amount was found to be substantially higher than those values allowed in the Drinking Water Directive.

(3) Whilst leaching was found to remove significant quantities of the calcium hydroxide with both distilled water and acidified water as the leaching agent, their effect on strength was insignificant. However, there was substantial increase in permeability of the concrete.

(4) The CRBA concretes could be used in structural applications if the leaching effect on water permeability was reduced by appropriate mix design and the use of pozzolans to modify the microstructure of the concrete.

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## 5 ACKNOWLEDGEMENTS

The authors are grateful to Mr. Mark Russell, the technician in Materials Analysis Lab for carrying out the DTA/TG analysis, and to Mr. Manus Carey who carried out ICP-MS analysis at The Queen's University Environmental Science and Technology Research Centre. The facilities for carrying out the experimental programme were provided by the School of Civil Engineering, which is gratefully acknowledged.

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# Fungal Colonization In The Paint Interfaces Of A Building Façade

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**Summary: The discoloration of painted façades is caused by a wide range of microorganisms, including phototrophs and fungi. The durability of paint films is affected by microbial growth and fungi are deemed to be especially important. In Brazil, the estimated cost of painting buildings is approximately 4% of the overall building cost, and it is necessary to re-paint approximately every 2 to 5 years, depending on other building characteristics. Biocides (chiefly algicides and fungicides) are usually included in paint formulations in an attempt to extend the period between repainting and the efficacy of these additives is tested in artificial conditions, in the laboratory. This study was carried out using modern surface analysis and traditional mycological techniques, in order to understand the fungal colonization of paint under real exposure conditions on a building newly painted with an acrylic paint with and without a biocide formulation. Observations with the low vacuum scanning electron microscope indicated that fungi can grow not only on the exposed surface, but also in the layers between old and new paint, and between the rendering mortar and paint layer. The latter surface was the most colonized interface on the façade of the building (constructed with aerated concrete blocks rendered with mortar). Cracks in the mortar rendering favored fungal growth on paint in this area. The exposed surface showed evident discoloration after 11 months of exposure. The fungal numbers on the painted surface decreased in the summer months, probably because of the intense solar radiation.**

**Keywords: discoloration, painted surfaces, Fungal Colonization**

## 1 INTRODUCTION

The development of discoloration on painted surfaces is a very common phenomenon in Brazil and in other countries. Sometimes this type of discoloration has been considered as fouling without any biological correlation, but it has been shown that it can be produced by different types of microorganisms (GARG et al, 1995; GAYLARDE & MORTON, 1999; KARPOVICH-TATE, 1990), among them autotrophic bacteria (those that do not need organic compounds as carbon source), heterotrophic bacteria (which need organic compounds as carbon source), cyanobacteria and algae (which use $CO_2$ as carbon source and light as energy source) and filamentous fungi (heterotrophic organisms composed of hyphae and reproductive structures, spores).

In order to reduce microbial colonization, the paint industries add biocides to the formulation (BESSEMS, 1996; BOGACKA, 1995; GAYLARDE, 1995; KAUFERS, 1995; MURRAY-SMITH, 1996).

The durability of paint films is affected by microbial growth and fungi are deemed to be especially important. In Brazil, the estimated cost of painting buildings is approximately 4% of the overall building cost, and it is necessary to re-paint approximately every 2 to 5 years, depending on other building characteristics. Biocides (chiefly algicides and fungicides) are usually included in paint formulations in an attempt to extend the period between repainting and the efficacy of these additives is tested in artificial conditions, in the laboratory. The mechanisms of deterioration include mechanical rupture, the production of metabolites, chiefly complex organic acids, and enzymatic activity. Water based paint is especially susceptible to fungi because of the cellulose added to maintain the liquid paint thickness (GILLATT & TRACEY, 1987).

We studied the sequence of fungal colonization on a newly painted building from the first week after application until one year of exposure, using modern techniques of surface analysis to try to understand the interaction between fungi and paint under real exposure conditions.

## 2 MATERIALS AND METHODS

### 2.1 Paint and biocide

An acrylic water paint with approximately 50% solids, with and without 0.25% biocide formulation (12% carbamate, 25% dimethyl urea and 4% isothiazolones) was used.

### 2.2 Selection and treatment of building façade and sample collection for isolation of microorganisms

A building about twenty years old, constructed with aerated concrete blocks rendered with mortar and painted, one year previously, with an acrylic paint was treated with approx. 2% hypochlorite for 15 minutes and then washed with a high pressure (11 MPa) water jet. After one week to allow thorough drying, the façade was painted with the selected paint in vertical strips, alternating paint with and without biocide. Four samples were taken from biocide-containing and 4 from non biocide-containing sites on this building at intervals (see results). Sampling was carried out by the carpet replica technique for fungi (SHIRAKAWA et al., 1998).

Samples for mycological analysis were cultured on Sabouraud dextrose agar and identified by cell and colony morphology (Barron, 1971; Ellis, 1993; McGinnis, 1980).

### 2.3 Paint sampling and surface analysis

Pieces of paint were collected using a chisel disinfected with alcohol and placed in sterile Petri dishes. For scanning electron microscopy under low vacuum (LEO 435 VP) and for confocal scanning laser microcopy (LSM 410 – Zeiss – with lasers 488 and 543, read in reflected mode), samples were examined directly without any treatment. For Scanning Electron Microscopy samples were coated with gold (Balzers SCD 050) and viewed in a Stereoscan 440 (Leika, Cambridge) after drying at 37°C for 48 hours.

## 3 RESULTS

### 3.1 Microorganisms

### 3.2 Fungi

The most frequent genus found over the whole year was Cladosporium, followed by the genera Alternaria, Epicoccum, Monascus, Curvularia, Nigrospora and Pestalotia. Other genera, Aspergillus, Arthrinium, Chaetomium, Didimostilbe, Fusarium, Monilia, Penicillium, Pleospora, Phoma, Pithomyces, Trichoderma, Trichotecium and Ulocladium, appeared in minor frequency.

Members of the artificial Order *Mycelia sterilia*, fungi that do not produce spores, were found at a high frequency throughout the year.

*Aureobasidium* showed an interesting behavior. This genus was found in the first four weeks (July and August of 2000), disappearing in the samples collected over September to April 2001, and appearing again in samples collected in May 2001. In July, one year after painting, this genus was found in high numbers of colony forming units, and dark discoloration was observed on the painted surface. This building façade received intense solar irradiation during the Summer (December to March), with air temperatures of 30 to 35°C during the day. In the literature *Aureobasidium sp* is considered the most aggressive fungus for paint and so its behavior is of obvious interest. Our results suggest that it is susceptible to UV and high temperatures.

### 3.3 Microscopical observations

### 3.4 Scanning electron microscopy

Samples of paint collected after 7 months of exposure showed that fungal colonization was chiefly located in regions where cracks were apparent. Figure 1 shows that in regions 1 cm from the crack, no fungal growth is evident, while directly over the cracked areas fungal growth can be clearly observed (Figure 2). The cracks in the paint were localized above fissures in the underlying the mortar. In addition, fungal growth itself seems to break up the paint film as observed in Figure 3.

Figure 4 (7 months of exposure) shows that fungi grow in the region with biocides, between the old and new paint layers, colonizing preferentially the old paint. Fungal growth and intense spore production is also observed at the interface between the old paint and the rendering mortar after this period of exposure (Figure 5); this colonization probably occurred before repainting. After 11 months of exposition fungi could be seen growing on the new paint with biocides in contact with old one (Figure 6).

### 3.5 Scanning laser confocal microcopy

Scanning laser confocal microscopy observation showed that fungi grow at different planes in the paint. Figure 7, a sample collected after 7 months of exposure, shows the interface between old paint and rendering mortar. Fungal hyphae are detected at 0 to 34.5 µm depth of confocal the plane, and are especially prevalent around 17µm. Together with quantitative culture data, results after the summer period the number of colony forming units increased greatly (Table 1), suggesting a re-growth, rather than merely a re-colonization from outside.

## 4 FINAL CONSIDERATIONS

These results show that fungal growth may be detected both at the interface between the old paint surface and the new paint layer and at the interface between the mortar and the paint. This shows that mortar may act as a reservoir for paint recolonization and that a combination of hypochlorite and high-pressure jet cleaning leaves a residue of viable fungal propagules. These sources of infection, together with air-borne propagules are important in the subsequent biodegradation of paint. Persistent propagules in the protected interfaces between old and new paint and under old paint, in contact with mortar or other substrates cause loss of adherence leading to spalling. Clearly preparation of surfaces prior to painting is of crucial importance in determining durability of new paint surfaces. This has long been recognized, but few studies have been directed to observing this aspect of paint durability. Indeed, we are not aware of any quantitative data published in the literature.

*Aureobasidium sp* showed a special behavior, being isolated from the surface in the first month of exposure and reappearing just after the end of the summer. This genus has been shown to be a primary colonizer also of leaves in both temperate and tropical climates; in both situations it disappears from the surfaces after the first month, reappearing 4-8 months later (LACEY, 1979).

The interactions between new and old paint layers, and between paint and mortar renderings, are important factors in determining fungal colonization. When applied to a porous mortar, the paint film could have pore size of 3 to 10 µm and some bigger. These pores, if they are interconnected, could allow fungal growth to spread readily through the paint film. This growth could be stimulated or inhibited by a range of factors. For example solar irradiation could induce the fungi to grow inwards and inhibit growth towards the outer surface. Although *Cladosporium sp* was the most frequent genus found throughout the year on this newly painted surface, tropical and semi-tropical environments, as found in São Paulo, where the study was carried out, favor growth of many types of fungi, including *Nigrospora sp.*, *Curvularia sp.* and *Pestalotia sp.* These should be considered for use in accelerated laboratory tests. Although after one year of exposure there still is a difference between the number of colony forming unit in regions with and without biocide, this difference has become non-significant and discoloration can be also observed by eye in painted regions containing biocide. Another building, with different substrate, painted with the same paint at the same time, has continuously showed less fungal colonization than that described above, even though the surrounding environment, much more wooded, is apparently more favorable to microbial growth. This is probably due to the influence of the substrate. It is intended to extend the study by exposing two different substrates painted with the same paint in three different climatic regions of Brazil.

**Table 1 – Counting of Colony Forming Unit of fungi after different intervals of exposure**

| Week | Without biocide (B-) | With biocide (B+) | Odds ratio (B+ < B-) |
|---|---|---|---|
| 1 | 0, 1, 4, 5 | 4, 4, 4, 5 | 1:3.12 |
| 2 | 3, 4, 4, 7 | 4, 5, 6, 8 | 1:4.83 |
| 3 | 12, 14, 16, 20 | 4, 9, 10, 12 | 68.9:1 |
| 4 | 11, 13, 20, 25 | 6, 9, 10, 15 | 16.5:1 |
| 9 | 11, 34, 41, 72 | 6, 10, 15, 26 | 16.5:1 |
| 13 | 12, 16, 19, 23 | 5, 11, 12, 13 | 34.0:1 |
| 18 | 17, 12, 17, 8 | 9, 9, 13, 7 | 4.83:1 |
| 22 | 19, 37,19, 19 | 28,20,16,32 | 1:1.26 |
| 26 | 24, 28, 21, 33 | 15, 20, 24,33 | 3.12:1 |
| 31 | 42, 23, 10, 15 | 22, 22, 20, 18 | 1:1 |
| 35 | 24, 16, 11, 11 | 10, 14, 12, 21 | 1.26:1 |
| 42 | 84, 48, 34, 51 | 32, 44, 48, 9 | 16.51:1 |
| 56 | 153, 56, 228, 47 | 88, 59, 63, 121 | 1:1 |

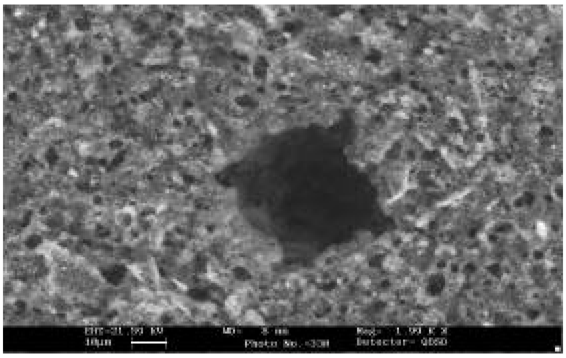

**Figure 1 – Low vacuum Scanning Electron Microscopy of samples collected from the façade painted with non-biocide-containing paint 1 cm from a crack over a fissure in the mortar. 7 months of exposure**

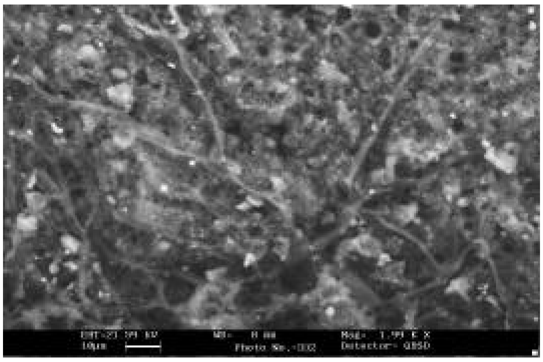

**Figure 2 - Low vacuum Scanning Electron Microscopy of samples collected from the façade painted with non-biocide-containing paint directly over the fissure in the mortar. 7 months of exposure**

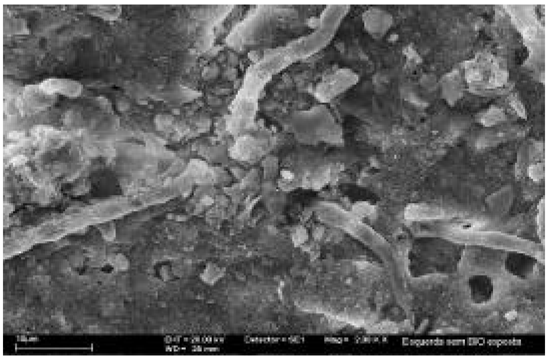

**Figure 3 – Scanning Electron Microscopy of samples collected from the façade painted with non-biocide-containing paint directly over the crack. In the area with fungal growth hyphae can be seen rupturing the paint film**

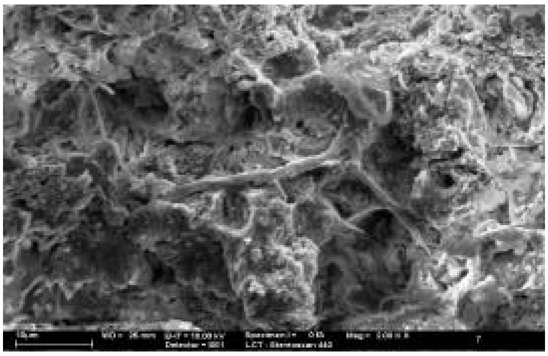

**Figure 4 - Scanning Electron Microscopy of samples collected from the façade. This image shows the old paint in contact with new paint in a biocide-containing region. 7 months of exposure**

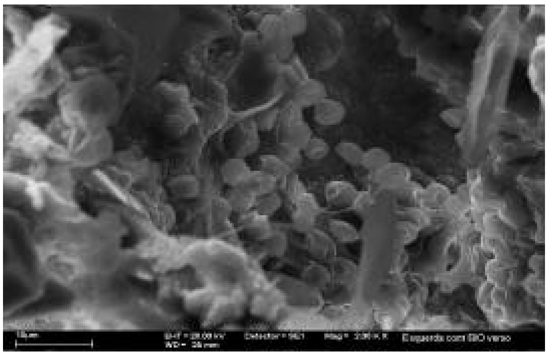

**Figure 5 - Scanning Electron Microscopy of samples collected from the façade. Region of old paint in contact with rendering mortar**

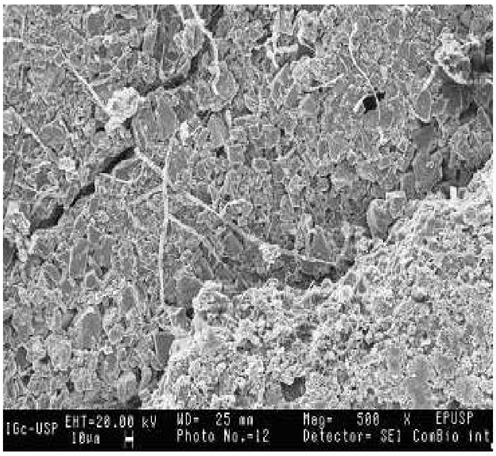


**Figure 6 - Scanning Electron Microscopy of samples collected from a biocide region of the façade after 11 months of exposure. Fungal growth is seen on the new paint film at the interface with the old paint**

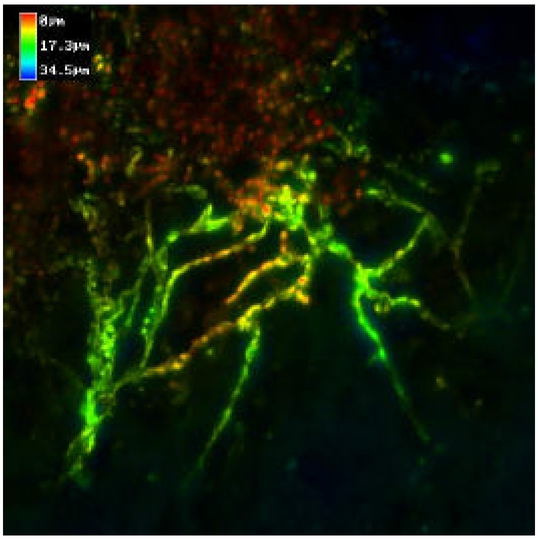


**Figure 7 – Scanning Confocal Laser Microscopy of sample collected after 7 months of exposure. Old paint in contact with mortar in region with fissure. Hyphae are seen not only on the surface in contact with mortar, but predominantly 17 micrometers of the confocal plane**

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# 5 ACKNOWLEGMENTS

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# Durability Investigations On Textile Reinforced Concrete

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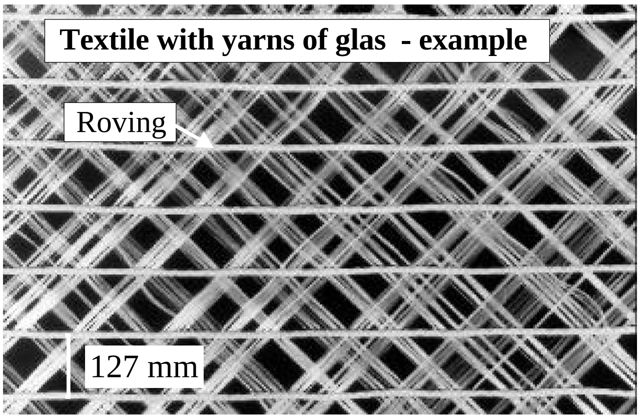
**Summary: Textile reinforced concrete is a new composite material, which offers many new and challenging applications due to its low weight and high load-bearing capacity.**

**The evaluation of the durability of textile reinforced concrete parts is a crucial aspect of the research into this new composite material. Climatic stresses of the composite material, e.g. due to humidity and temperature, result in changes in the bond and strength characteristics of the yarn/concrete system. In order to detect these changes, the Institute for Building Materials Research in Aachen uses a variety of testing procedures, which are often combined. This paper explains the TSP test (dog-bone shaped specimen). This test shows the changes in the stress-strain behaviour, the cracking image as well as the maximum yarn tensile strength of specimens, which have been reinforced with yarn or textiles and are stressed in one axis due to climatic stresses. The results of the TSP test shows a clear functional loss of these specimens, which have been stored in warm water for several days. The collapse load and the ultimate strain as well as the number of cracks are reduced due to the storage in warm water. A chemical attack as well as a mechanical attack can be regarded as the causes for the loss of functionality of the reinforcement. Both damaging mechanisms will be discussed. With help of the described structural examinations the pore area in the contact zone of yarn/concrete after a climatic stress is detected. The fact that the pore area in the contact zone after the storage in warm water is reduced, compared to a storage at 20 °C and 65 % relative humidity, indicates a mechanical stress of the reinforcement.**

**Keywords: Durability, Accelerated Ageing, Textile Reinforced Concrete, Tensile Test, Bond Behaviour**

## 1 INTRODUCTION

Textile reinforced concrete represents an interesting new construction material, which offers several advantages compared to steel or fibre reinforced concrete. These advantages dominate in the field of applications, where thin-walled, structural elements with a high load-carrying capacity are necessary. The following materials can be used as a reinforcement: glass rovings, carbon rovings or polymer rovings. These rovings textiles as shown in fig. 1, left, are fabricated and placed in the tension area of the component. The employed rovings are multifilament yarns, consisting of more than 500 filaments with diameters between 12 and 30 µm. A roving with 2400 tex (describing fineness: 1 tex = 1 g / 1000 m) has about 2400 Filaments. Figure 1 shows on the right side a roving which is embedded in concrete.



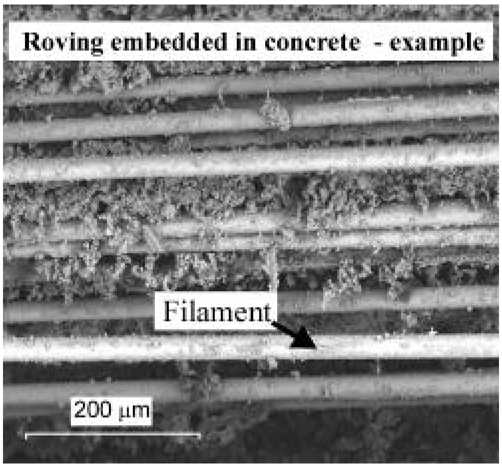




**Figure 1. Examples for a textile used as a reinforcement of concrete and for a roving, which is embedded in concrete**

One aim of the research project SFB 532 "Textile Reinforced Concrete – Basics of a New Technology" placed at the Technical University of Aachen (financed by the central public funding organisation for academic research in Germany, DFG) is to investigate the durability of this construction material.

For the assessment of the durability of the composite material the following two aspects should be taken into consideration: The types of climatic stresses that are to be applied as well as the determination of the changes in the bond and strength characteristics that result from the stress. Within this paper the climatic stress is confined to a laboratory stress with different temperature and humidity values. The storage of the composite components in a water bath at a hightened temperature leads to an accelerated ageing process as it has been shown, amongst others, by Litherland et al (1981). In order to be able to determine the changes of the bond and strength characteristics which result from the climatic stresses, different kinds of examinations are necessary. The SIC test (strand in cement) allows conclusions concerning the change of the maximum tensile strength of a roving, that is embedded in the concrete, after different climatic stresses. This test was and is used extensively to examine different material combinations and compare them with each other (Raupach and Brockmann (2001)). The changes in the pull-out behaviour of a roving in concrete due to climatic stresses is examined with the help of two-sided pull-out tests (Raupach and Brockmann (2001)). The TSP test carried out on dog-bone shaped specimens allows conclusions concerning the changes in the stress-strain behaviour, the cracking image as well as the maximum yarn tensile strength of textile reinforced specimens under uniaxial stress due to climatic stress. This paper explains the TSP test. Some test results are presented and discussed.

The changes in the behaviour of the composite material, which have been determined in the tests, have two causes that are described in other papers in different ways (e.g. Litherland et al (1981); Majumdar et al (1980); (Yilmaz and Glasser (1992)). Firstly the glass is chemically attacked by the high alcalinity of the binding agent and secondly it is mechanically attacked by hydration products. The hydration products lead to a compression of the pore area next to the roving. This compression heavily restricts the movability and thereby the elongability of the individual filaments. The loss of elongability leads to the inability or the reduced ability of the roving to contribute to the load carrying. Purnell et al (2001) are convinced that the local points of flaws of the glass, the size of which is about 0.2 µm (Purnell (1998)) and which are due to the manufacturing process, are the points of attack by chemical and mechanical forces. The two damage mechanisms are discussed extensively in the course of this paper. At the end of the paper an examination will be introduced, which can be used to determine the compression of the pore area around the roving after different kinds of storage.

## 2 SETUP AND IMPLEMENTATION OF THE TSP TEST

The TSP test examines dog-bone shaped specimens (TSP), which have been reinforced with rovings or textiles, after different climatic stresses with the help of tensile tests.

Figure 2 shows the manufacturing of the specimens in a steel formwork. The rovings are evenly distributed over a specimen width of 60 mm. At first the rovings are mounted on a frame. Then they are collected and fixed at the ends with steel clamps and an elastomer and inserted in the formwork. In order to fixate their position, the rovings are tensed against the formwork with the clamp. The applied tensile force is far below the yielding point of the material. In order to seal the front sides of the formwork, polymer rods are inserted into the respective relief in the formwork. These rods are coated with a sealing material. The concrete is filled into the formwork over the sample thickness of 7 mm and compacted by vibration.

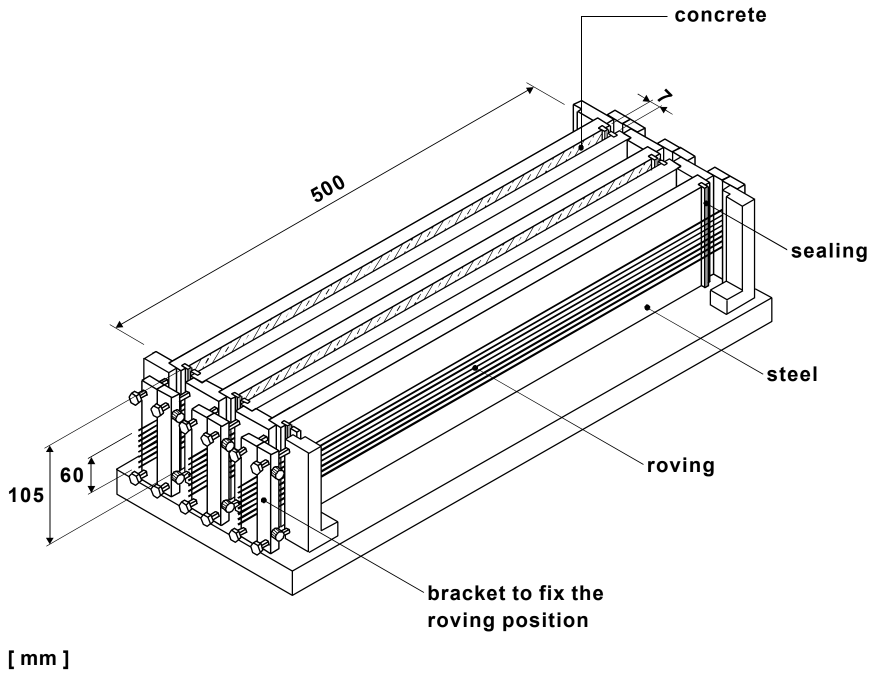



**Figure 2. Manufacturing of the TSP specimens in a steel formwork**

The specimens are manufactured with a constant width of 105 mm. After that the specimens are machined with the help of a concrete mill. In order to increase the manufacturing accuracy the milling head moves along a CNC manufactured aluminium template. The geometry of the dog-bone shaped specimens can be seen in figure 3. In the centre area of the specimen, the specimen width is a constant 60 mm over a length of 220 mm. This leads to an homogenious stress distribution in the middle part of the specimen.

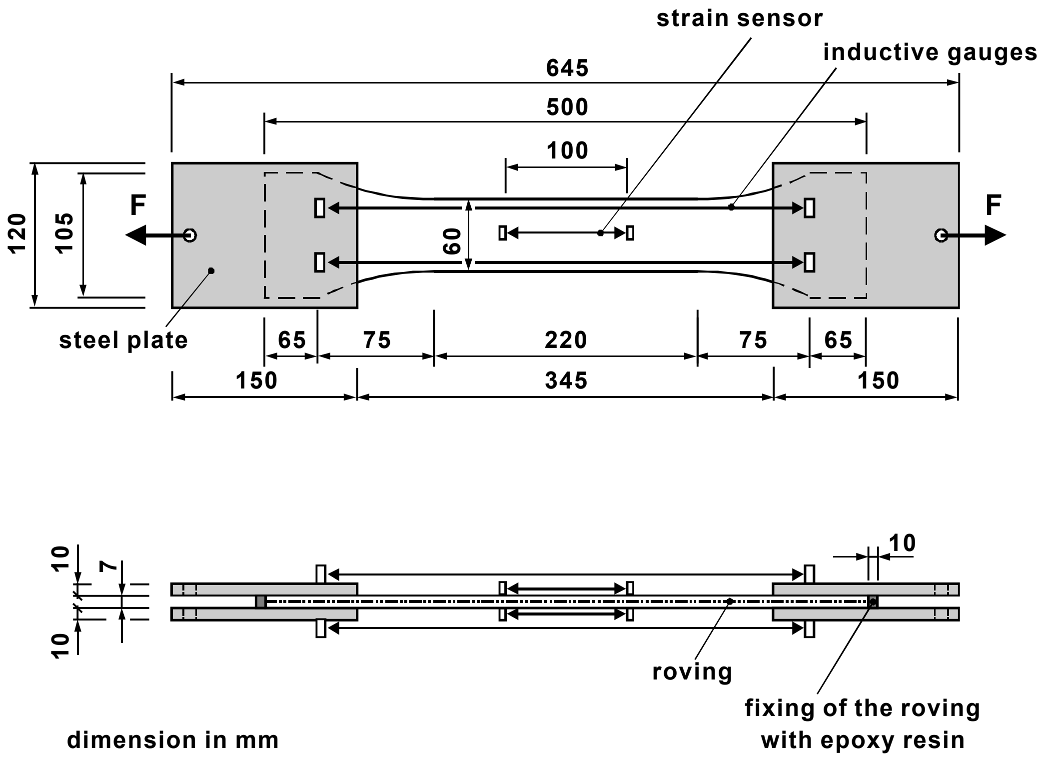



**Figure 3. layout and dimensions of the TSP-specimens**

For the application of the tensile force, holed steel plates are glued to the specimen. The force application is done via a pin in the steel plate. From there the force application continues via the glue joint into the tensile sample. Thereby an unwanted transversal pressing is avoided. The TSP samples are charged with a speed of 0.5 mm/min until the failure occurs. The change of length is measured over the complete sample length with inductive gauges, which are fixed to the steel plates. The shear deformation in the glue joint has been determined. Its value can be neglected for the test evaluation as a thin glue joint is used,

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which has a high stiffness. The acting strain until the first cracks develop is additionally determined over 100 mm with a DD1 distance sensor, which is centred over the sample. The rovings, which are sticking out of the sample body, are glued with epoxy resin. This way the individual filaments are fixated at the end of the sample. When stress is applied, individual filaments are unable to slip towards the centre of the sample from the sample edge. Therefore the rovings can reach the maximum tensile strength. Furthermore the epoxy resin glue of the outside of the yarn prevents any stress medium (e.g. water) from getting into the sample body via the yarn. This test allows conclusions concerning the change in the stress-elongation behaviour, the cracking image as well as the maximum yarn tensile strength after climatic stresses.

## 3 TSP TEST RESULTS AND DISCUSSION

For the results shown in this paper, glass rovings manufactured by the company NEG with 2400 tex have been used. The TSP samples were reinforced with 8 rovings. With a sample thickness of 7 mm this corresponds to a reinforcement degree of 1.7 vol. %. Preliminary investigations showed, that this is the highest possible reinforcement degree under the constrain that each roving is completely coated by concrete and that there is no overlapping of rovings inside the specimen. The concrete mixture (table 1) that has been used, was developed by the Institute for Building Materials Research as a textile concrete (Brameshuber and Brockmann (2001)). Hereby the complete penetration of the textile reinforcement demands a high flowability as well as a small grain size of the aggregate. Due to the simultaneous addition of silica fume and coal fly ash the contents of alkali ions and calcium hydroxide is reduced compared to pure Portland cement mixtures. The pH value of this mixture is 13.5.

**Table 1. Composition of the fine concrete mixture**

| *Binder system* | *Type of cement* | *Cement content* | *Additives* | | | | *Binder content* | *w/b-ratio* | *max. grain size* |
|---|---|---|---|---|---|---|---|---|---|
| | | | *Fly ash* | *Silica fume* | *Water reducer* | *Stabilizer* | | | |
| | | Kg/m$^3$ | | | - | | kg/m$^3$ | - | mm |
| OPC | CEM I 52,5 | 490 | 175 | 35 | 1[2] | - | 700 | 0.4 | 0.6 |

[2]: Mass percentage in relation to cement content

After the samples have been manufactured they are all stored in a humidity cupboard at 23 °C and 95 % relative humidity for 28 days followed by the climatic stress. Therefore it can be assumed that at the time of the climatic stress the hydratation has been more or less accomplished. Figure 4 (top left) shows three tensile test curves after the 28 days storage period in the climatic chamber. The force is always drawn over the elongation. The tests shows a low deviation. After a linear elongation the initial crack appears at approx. 1.9 kN. The area of the crack development with crack distances of about one centimeter ends at approx. 2.5 kN. Thereafter the load carrying happens via the 8 yarns until the reinforcement fails at an average value of 4.6 kN. Figure 5 shows the destroyed sample on the left hand side.

In the top right part of figure 4 the influence of a 3 days water storage at 50 ° is shown. The initial cracking load is risen to an average value of 2.3 N due to the warm water storage. After the initial cracking these samples also show an evenly distributed cracking image. The sample bodies fail at an average value of 3.5 kN.

After a warm water storage at 80 °C for three days (figure 4, bottom left) an evenly distributed cracking image also appears at an average value of 2.6 kN. After the cracking image has been accomplished, there is also a load carrying capacity of the yarns but the carrying ability of the yarn decreased to an average value of 3.1 kN.

The gradient of the curves, which are nearly linear, in the after-crack area is up to approx. 8 % lower for the samples that have been stressed by warm water. The gradient decreases for increasing temperature of the water storage. The curve in this area is determined by the volume and the modulus of eleasticity of the yarn. Purnell (1998) comes to the conclusion that the smaller gadient relates to a loss of yarn volume due to the built-up of flaws in the glass after the chemical attack.

After the sample has been stored in warm water at 80 °C for seven days (figure 4, bottom right), it fails abruptly at a rectangle to the reinforcement (figure 5, right) at an average value of 2.6 kN, i.e. on the level of the initial crack. The glass reinforcement is no longer able to contribute to the load carrying.

Two causes can be named for the loss of functionality of the reinforcement: a chemical attack on the glass and the resulting loss of yarn volume as well as a mechanical attack due to hydratation products and the resulting loss of ductility or transversal shearing respectively on the roving. These two damaging mechanisms will be explained in detail in the following chapter.

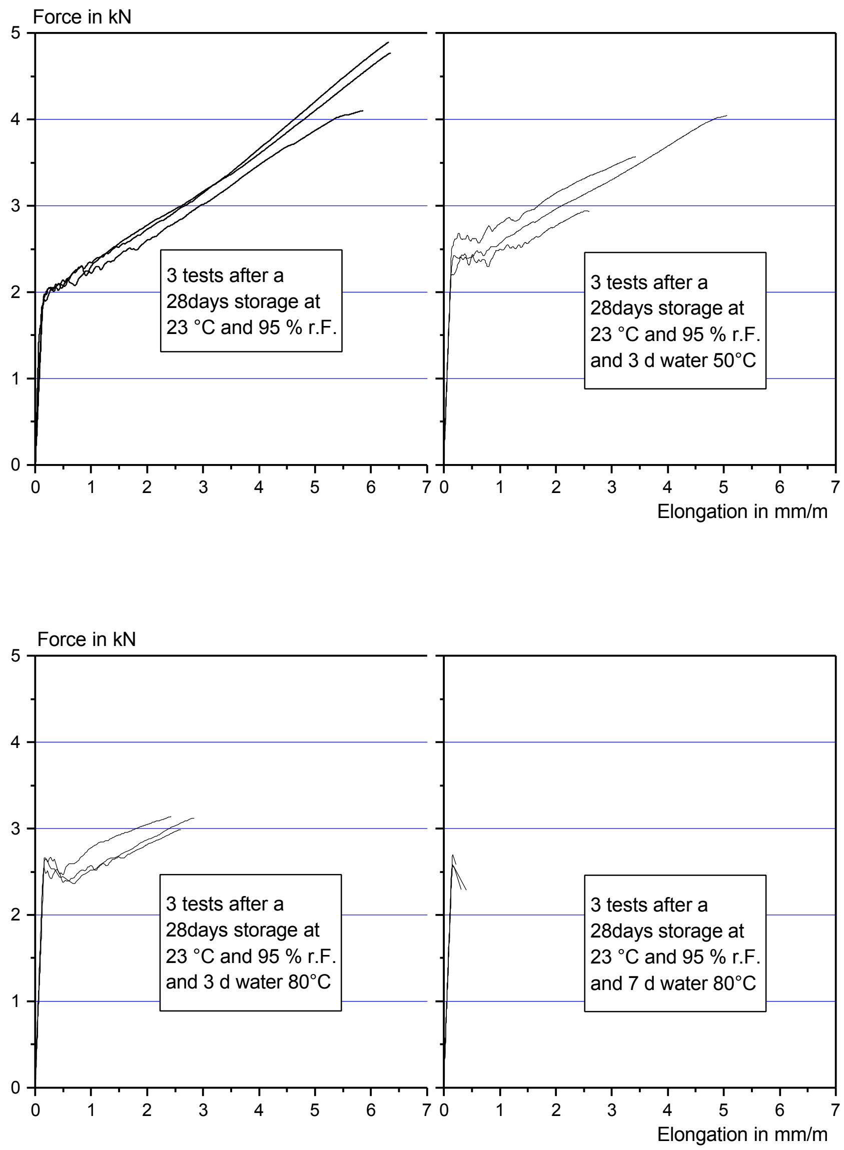


**Figure 4. TSP tests with NEG rovings 2400 tex and concrete after different climatic stresses**

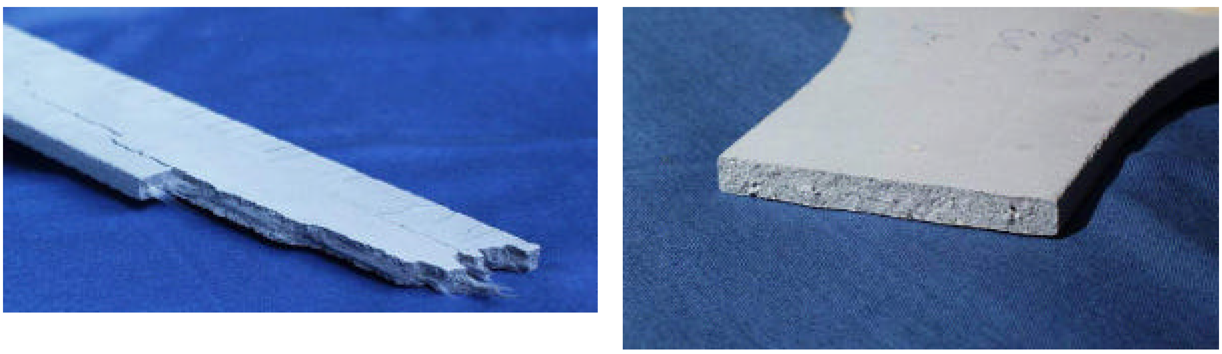

**Figure 5. Failure of TSP samples after 28 days storage at 23 °C and 95 % r. h. (left) and after 28 days storage at 23 °C and 95 % r. h. and 7 days in water at 50 °C**

Figure 6 allows an overview of the decrease in the ultimate loads and the ultimate strain of the TSP samples, which have been stored under different conditions, in the tensile test. The 28 days value (storage time under climatic stress: 0d) after a storage in the climatic chamber is 4.6 kN and is regarded as the start value. The appliable force decreases linearly between 3 and 14 days at a 50 °C water storage. It levels at a value of approx. 0.5 kN above the initial cracking load. However the elongation of rupture has decreased further after a 28 days storage in warm water. This loss of force has a tendency to stagnate between day 14 and 28 of the water storage at 50 °C and will be investigated further in a new test series. The appliable yarn tensile force of the samples, which have been stored in water at 80 °C for 7 days (or 14 days respectively), is below the initial cracking force and therefore cannot be detected with this test. The reinforcement degree of the samples cannot be increased due to the manufacturing process and the transfer of the test results into reality becomes more and more difficult with an increasing water temperature, especially for concrete the basis of which is not Portland cement or blast furnace cement. Therefore future TSP tests will be increasingly performed after a water stress at 50 °C.

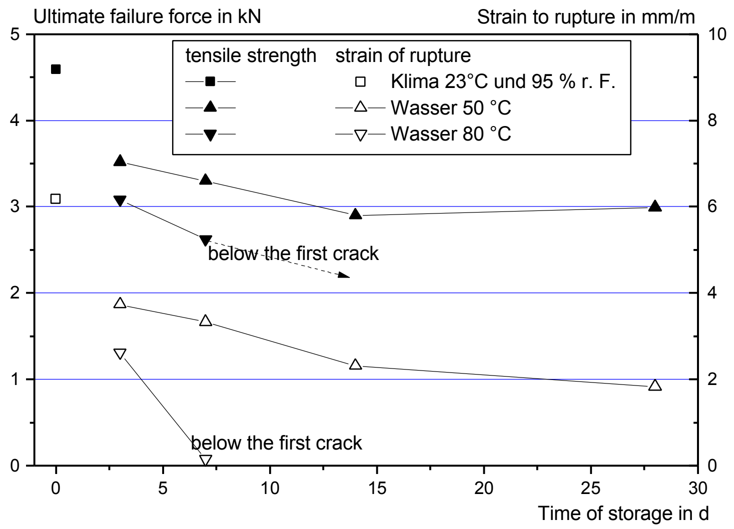


**Figure 6. Rupture forces and respective elongations in the TSP test after different kinds of storage**

## 4 DAMAGING MECHANISMS

The results of the TSP tests shows a clear loss of strength of the composite material with an increasing water temperature and storage time. Two causes can be named for the functional loss of the reinforcement:

- A chemical attack on the glass: Hereby the silicon oxide ($SiO_2$) network is nucleophilicly attacked by a hydroxyl ion. This reaction is self-generating due to follow-on reactions, whereby a complete dissolution of the glass is possible. In addition to the parameters that correspond to the corrosion medium, the composition of the glass is of crucial importance for the speed of the reaction (Czymai (1995)). The dissolution of the glass leads to a loss of roving volume, which again equals a reduction of the strength.
- A mechanical attack on the glass: Hydration products, mainly calcium hydroxide, increasingly build up along the border lines of filament and matrix. Since the concrete penetrates the roving in an uncontrolled manner during the concreting process (as shown in figure 7), a large area is available for the build-up of hydration products. This area is compressed which results in a loss of ductility. Another aspect is the transversal shearing of the hydratation products on the filaments. The glass, which is sensitive to transversal forces, therefore also fails at an early stage.

The TSP-Test cannot achieve a separation of these two damaging mechanisms. According to the authors knowledge no researcher has so far succeeded in describing the damaging mechanisms separately. This will be the aim of future research at the Institut for Building Materials Research in Aachen. In this respect the following two points demand a closer study:

- The rovings, which are embedded in the concrete, show inclusions of air and binding agents. These lead to a very uneven distribution of filaments in each sample cross section as shown as an example in figure 7. In the course of their examinations Brameshuber and Banholzer (2001) also found a high number of flaws in a concreted roving. Due to dispersion etc. these flaws cause problems with the test evaluation.
- During the manufacturing process of the rovings a size is applied as a protection and for improved treatment. This polymer coating, which is very thin and does not cover the roving completely, varies according to the manufacturer. The size leads to certain conditions in the contact zone roving/concrete, which have not been identified so far. Therefore a swelling of the size can lead to an increase in the porosity in the contact zone.

First examinations, which investigated the chemical attack and the mechanical attack separately, have been documented by Brockmann et al (2001).

### 4.1 Structural examinations in the contact zone roving/concrete

At the present time the Institute for Building Materials Research in Aachen investigates the possibility of a matrix compression in the contact zone roving/concrete as well as a possible resulting mechanical attack on the reinforcement. These tests are performed with help of the image analysis evaluation of scanning electron microscope images.

For these tests a glass roving is embedded in the centre of a concrete body (length 40 mm, width 10 mm, thickness 10 mm). The roving that sticks out of the concrete body is protected from the attacking medium by a silicon hose. The samples are stored under different conditions (variation of temperature and humidity). For the preparation of the samples the hydration is stopped with the help of acetone. After the samples have been dried for two days at 50 °C, they are impregnated with epoxy resin under a vacuum. After the impregnation with epoxy resin and a subsequent application of a pressure of 5 bar, the samples are sawn transversally to the roving and polished. Element contrast images are taken of the plan view of the cross section in the scanning electron microscope. Those areas of the element contrast images, which appear black, are pores, filled with epoxy resin. These images are therefore very helpful to detect pore areas in the matrix of the contact zone and the yarn area. This is done with help of an image analysis system from the company SIS. This process is shown in figure 7.

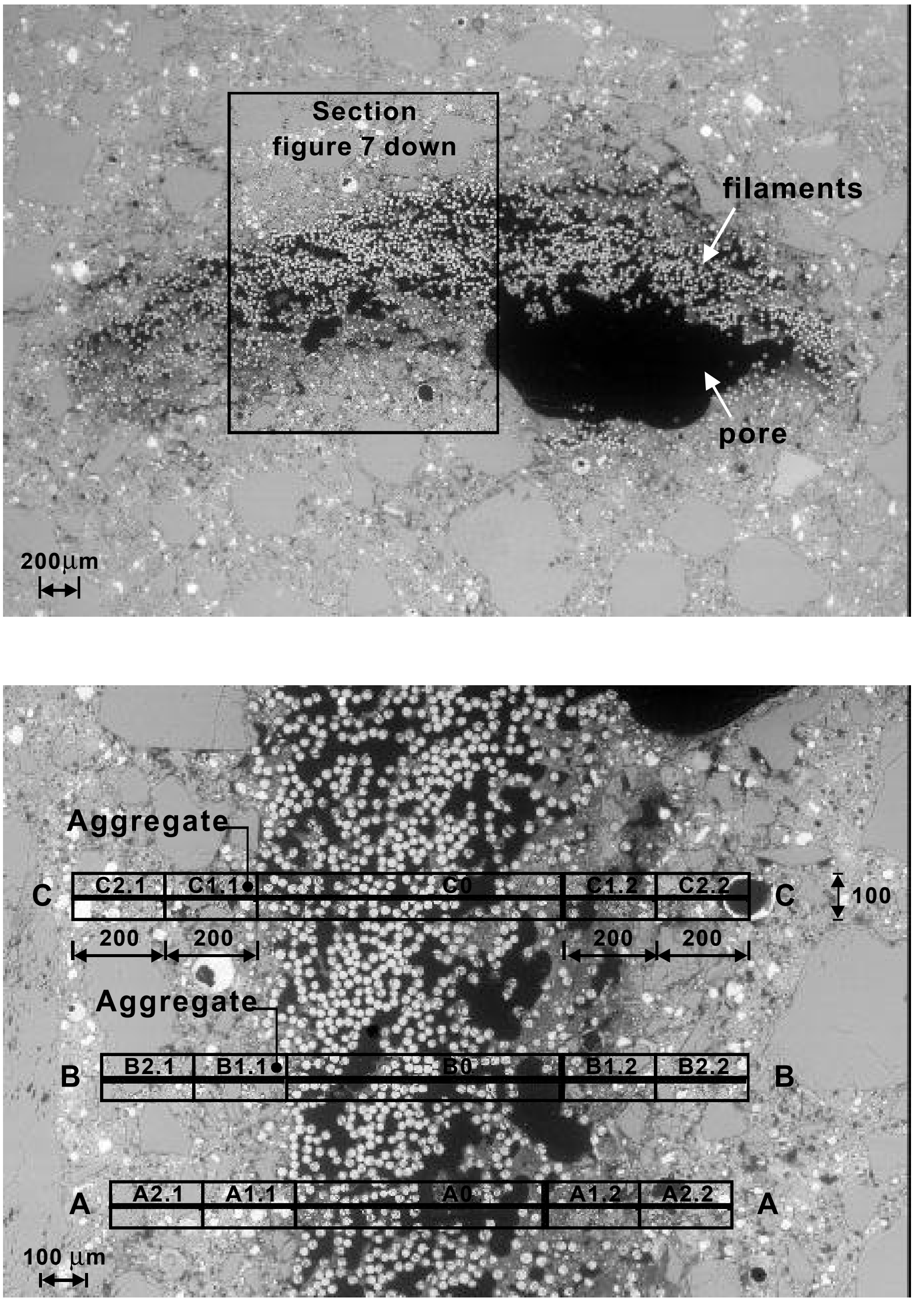


**Figure 7: Element contrast image of a sample and determination of the pore areas on an image section**

In order to investigate the pore distribution in the contact zone and the concrete as well as their changes due to climatic stresses, despite of the uneven filament distribution, the following procedure is applied: Starting with the total view of a roving cross section (figure 7, top), afterwards element contrast images of small parts of the yarn cross sections are taken with a magnification factor of 60. 3-4 axial sections are placed in these images (figure 7, bottom). Outside the rovings these sections are separated into areas of 100x200 µm². A separation between roving and contact zone is made at the aggregate grain, which is nearest to the roving. The pore area of the individual rectangles is determined via an image analysis evaluation and an average value is found via the large number of axial sections. If the number of observed areas in the contact zone and the fine concrete is sufficient (>15 areas of 100x200 µm), conclusions concerning the pore distribution according to different storage types become possible. Table 2 shows the average pore area in % of the examined areas in the fine concrete and in the contact zone roving/concrete after different storage types. Rovings of two different manufacturers were examined.

The samples, which have been reinforced with CemFil 2400 tex, do not show any changes of the pore area in the concrete when they are stored in a 56 days climatic storage or a water storage at 50 °C respectively. However the pore area in the contact zone has decreased from 12 % (56 d at 20 °C/60 % r.h.) to 5 % after a 112 days water storage at 50 °C. For the samples, which have been reinforced with NEG 2400 tex (this yarn has also been used for the TSP samples), the determined pore area is larger than for the samples, which have been reinforced with CemFil. This may be explained with the smaller content of interlaced material in the NEG size (Hoffman and Höcker (2001)). The increase of the pore area in the fine concrete for the samples, which have been reinforced with NEG, indicates the large area of influence of the sized roving. In the future the pore area of the fine concrete will additionally be determined at an increased distance to the reinforcement. It is also planned to examine several cross sections at one sample body, in order to determine the dispersion range of the respective pore area via the length of the sample.

**Table 2. Image analysis determination of the pore area in % in the concrete and in the contact zone roving/concrete**

| *Roving* | *Storage* | *Averaged pore area in % from a section of 100x200 µm* | |
|---|---|---|---|
| | 28 d at 23°C / 95 % r. h. + | concrete | Contact zone roving/concrete |
| CemFil 2400 | 56 d at 20°C / 65 % r. h. | 3 | 12 |
| | 56 d in water at 50 °C | 3 | 5 |
| NEG 2400 | 112 d at 20°C / 65 % r. h. | 7 | 15 |
| | 112 d in water at 50 °C | 4 | 7 |

These structural examinations will be carried out parallel to further detailed examinations, in order to guarantee a proper interpretation of the test results by combining extensive test results. At the end of this project it shall be possible to view the damaging mechanisms and their influence on the durability of the composite material separately.

## 5 SUMMARY

The TSP test allows conclusions concerning the change in the stress-strain behaviour, the cracking image as well as the maximum yarn tensile strength of yarn or textile reinforced components under uniaxial load due to climatic stress. The results of the TSP tests shows a clear functional loss of the composite material after storation in warm water for several days. The ultimate load and the ultimate strain as well as the number of cracks are reduced due to the storage in warm water. After the specimen has been stored in water at 80 °C for seven days, it fails abruptly when the tensile strength of the concrete is reached, the glass reinforcement does not contribute any longer to the load carrying capacity. The cause for the loss of load carrying capacity can be a chemical attack as well as a mechanical attack on the glass reinforcement or a combination of both attacks. A separate view and classification of these two damaging mechanisms can only be done with extensive investigations. A part of the necessary investigations are the structural examinations shown in this paper, which determine the pore area in the contact zone and in the concrete after different climatic stresses of the composite material. The pore areas are determined with the help of image analysis evaluation of scanning electron microscope images. These pore areas decrease in the contact zone roving/concrete due to a water storage at 50 °C as compared to a climatic storage at 20 °C and 65 % relative humidity. The reduction of the pore area leads to a compression of the matrix in this area and therefore results in a loss of ductility of the reinforcement.

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# Strength Development And Polymer Durability In Autoclaved Polymer-Modified Mortars

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**Summary**: **In general, the application of a combined wet/dry cure and a dry cure to polymer-modified mortars causes no degradation of the polymer films formed in them. However, the application of autoclaving to such mortars is anticipated to cause the degradation of the polymer films due to exposure to high temperature and pressure. The strength development of the autoclaved polymer-modified mortars may depend largely on the type of polymer. Polymer-modified mortars using polymer dispersions such as a styrene-butadiene rubber (SBR) latex, a poly(ethylene-vinyl acetate) (EVA) emulsion and a polyacrylic ester (PAE) emulsion for polymeric admixtures, and ground granulated blast-furnace slag (slag) and high-purity silica (silica) as siliceous admixtures are prepared with a mass ratio of binder (a mixture of ordinary portland cement and slag or silica) to Toyoura standard sand 1 : 3, slag or silica contents of 0, 30, 40 and 50%, and polymer-binder ratios of 0, 5, 10, 15 and 20%, cured under autoclaving at a temperature of 180? and a vapor pressure of 1.01 MPa, and tested for flexural and compressive strengths. The polymer films prepared with the same polymer dispersions are exposed to autoclaving under the same conditions, and subjected to tensile test and infrared spectroscopy. The durability of the polymer films is evaluated from the viewpoints of the deteriorated basic physical properties and changed infrared spectra. As a result, regardless of the slag or silica content, the flexural and compressive strengths of SBR-modified mortars tend to increase with an increase in the polymer-binder ratio, while those of EVA-and PAE-modified mortars decrease with increasing polymer-binder ratio. The application of autoclaving to the polymer films under saturated $Ca(OH)_2$ solution immersion causes no degradation for SBR film and a significant degradation due to the saponification of the polymers for EVA and PAE films. Accordingly, autoclaving does not significantly degrade SBR-modified mortars but markedly degrades EVA-and PAE-modified mortars by the saponification of the polymers.**

**Keywords. Autoclaving, Polymer-Modified Mortar, Strength Development, Polymer Durability, Polymer Film**

## 1 INTRODUCTION

In polymer-modified mortars, cement hydration and polymer film formation proceed simultaneously, and co-matrix phases are formed by both processes. Cement hydrates are reinforced with the network structures of the polymer films in the co-matrix phases. Therefore, the morphology and properties of the polymer films strongly affect the properties of the polymer-modified mortars. The application of a combined wet/dry cure and a dry cure to the polymer-modified mortars generally causes no degradation of the polymer films formed in them. However, the application of autoclaving is anticipated to cause degradation of the polymer films due to exposure to high temperature and pressure .The strength development of the autoclaved polymer-modified mortars may depend largely on the type of polymer. The purpose of this investigation is to determine the strength development and polymer durability in the autoclaved polymer-modified mortars. In the present paper, polymer-modified mortars using three polymeric admixtures and two siliceous admixtures were prepared with various siliceous admixture contents and polymer-binder ratios, cured under autoclaving, and tested for strength. The polymer films prepared with the same polymeric admixtures were exposed to autoclaving under the same conditions, and polymer durability was evaluated in terms of the deteriorated physical properties and changed infrared spectra.

## 2 MATERIALS

### 2.1 Cement

Ordinary portland cement as specified in JIS(Japanese Industrial Standard) R 5210(Portland cement) was used as a cement.

### 2.2 Admixtures

#### 2.2.1 Siliceous admixtures

Ground granulated blast-furnace slag (slag) and high-purity silica (silica) were used as siliceous admixtures. The properties of the siliceous admixtures are given in Tables 1 and 2.

#### 2.2.2 Polymeric admixtures

Polymer dispersions used as polymeric admixtures were a styrene-butadiene rubber (SBR) latex, a poly(ethylene-vinyl acetate) (EVA) emulsion and a polyacrylic ester (PAE) emulsion. The properties of the polymer dispersions are listed in Table 3. Before mixing, a silicone emulsion-type antifoamer was added to the polymer dispersions in a ratio of 0.7% of the silicone solids of the antifoamer to the total solids of the polymer dispersions.

**Table 1. Properties of ground granulated blast-furnace slag.**

| Density (g/cm$^3$) | Blaine specific surface (cm$^2$/g) | Activity index (%) | | |
|---|---|---|---|---|
| | | 7d | 28d | 91d |
| 2.91 | 10070 | 128 | 115 | 106 |

| Chemical compositions (%) | | | |
|---|---|---|---|
| MgO | $SiO_2$ | $SO_3$ | $Cl^-$ |
| 5.58 | 33.5 | 0.12 | 0.003 |

**Table 2. Properties of high-purity silica.**

| Density (g/cm$^3$) | Blaine specific surface (cm$^2$/g) | Average particle size (µm) | $SiO_2$ (%) |
|---|---|---|---|
| 2.65 | 5060 | 7.6 | 99.90 |

**Table 3. Properties of polymer dispersions.**

| Type of polymer dispersion | Density (20? , g/cm$^3$) | pH (20? ) | Viscosity (20? , mPa? s) | Total solids (%) |
|---|---|---|---|---|
| SBR | 1.02 | 9.4 | 64 | 44.7 |
| EVA | 1.07 | 5.2 | 1218 | 44.0 |
| PAE | 1.06 | 9.8 | 49 | 47.0 |

#### 2.2.3 Fine aggregate

Toyoura standard sand as specified in ex-JIS R 5201(Physical testing methods for cement) was used as a fine aggregate.

## 3 TESTING PROCEDURES

### 3.1 Tests of strength development

#### 3.1.1 Preparation of specimens

Before mixing polymer-modified mortars, the respective binders were prepared by blending cement and siliceous admixture (slag or silica) by use of a small ball mill for 4h. The cement: siliceous admixture (slag or silica) ratios (by mass) of the binders were 100: 0, 70: 30, 60: 40 and 50: 50, corresponding to slag or silica contents (SL or Si) of 0, 30, 40 and 50%. According to JIS A 1171 (Test methods for polymer-modified mortar), polymer-modified mortars were mixed with a mass ratio of the binder to fine aggregate 1: 3, polymer-binder ratios (P/B, calculated on the basis of the total solids of polymer dispersions) of 0, 5, 10, 15 and 20%, and their flow were adjusted to be constant at 170±5 in the water-binder ratio range of 50.0 to 78.0%. Beam specimens 40×40×160mm for strength test were molded, precured at 20? and 80%(RH) for 48h, and then subjected to autoclaving at a maximum temperature of 180? and a pressure of 1.01MPa for 3h.

#### 3.1.2 Flexural and compressive strength tests

The autoclaved beam specimens were tested for flexural and compressive strengths in accordance with JIS A 1171.

### 3.2 Tests of polymer durability

#### 3.2.1 Preparation of polymer films

Polymer dispersions filtered with wire gauze with a nominal size of 74µm were poured in the molds 200x200x1mm set on polyester-sheeted glass plates placed horizontally, and dried with a hood to prevent air flow at 30? for 1d to make polymer films. After drying, the polymer films were removed from the polyester-sheeted glass plates, and dried further at 20? and 60%(RH) for 5d.

#### 3.2.2 Physical tests

Polymer films were tested for minimum film-forming temperature according to JIS K 6828 (Testing methods for synthetic resin emulsions), for glass transition temperature according to JIS K 7121(Testing methods for transition temperatures of plastics), and for tensile behavior in accordance with JIS K 6251 (Tensile testing methods for vulcanized rubber).

#### 3.2.3 Degradation tests under autoclaving

Polymer films were subjected to autoclaving under saturated $Ca(OH)_2$ solution immersion or nonimmersion at a maximum temperature of 180? and a pressure of 1.01MPa for 3h. The polymer films before and after autoclaving were tested for tensile strength and elongation in accordance with JIS K 6251. The samples extracted with toluene from the polymer films before and after autoclaving were analyzed at wavenumbers of 600 to 4000$cm^{-1}$ by infrared spectroscopy.

## 4 TEST RESULTS AND DISCUSSION

### 4.1 Strength development

Figures 1 and 2 show the effects of slag and silica contents on the flexural and compressive strengths of autoclaved polymer-modified mortars with polymer-binder ratios of 0, 5, 10, 15 and 20%, respectively. Figures 3 and 4 illustrate the effect of polymer-binder ratio on the flexural and compressive strengths of the autoclaved

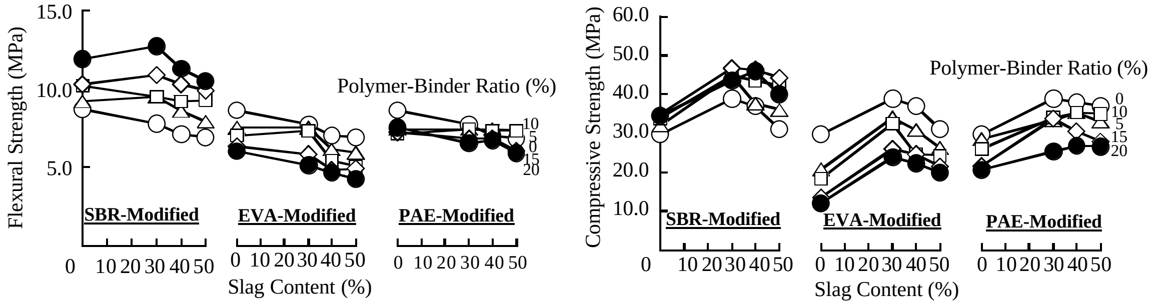


**Figure 1. Slag Content vs. Flexural and Compressive Strengths of Autoclaved Polymer-Modified Mortars with Polymer-Binder Ratios of 0, 5, 10, 15 and 20%**

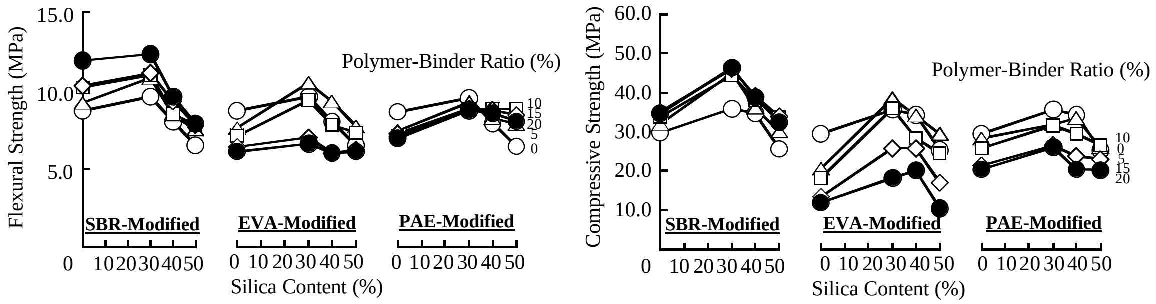


**Figure 2. Silica Content vs. Flexural and Compressive Strengths of Autoclaved Polymer-Modified Mortars with Polymer-Binder Ratios of 0, 5, 10, 15 and 20%**

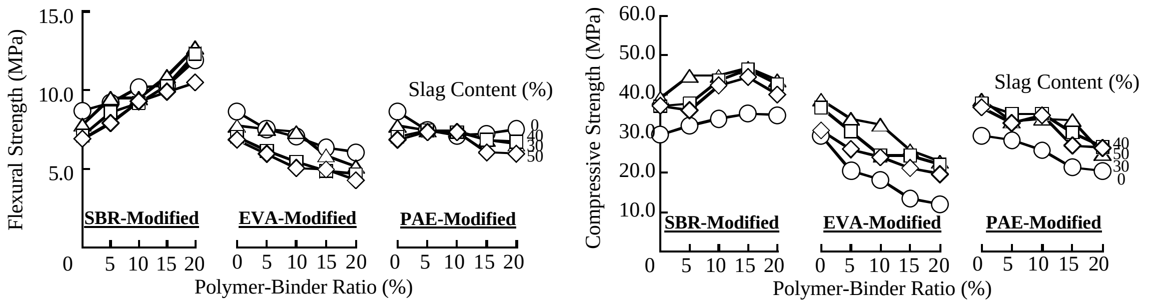


**Figure 3. Polymer-binder ratio vs. flexural and compressive strengths of autoclaved polymer-modified mortars with slag contents of 0, 30, 40 and 50%**

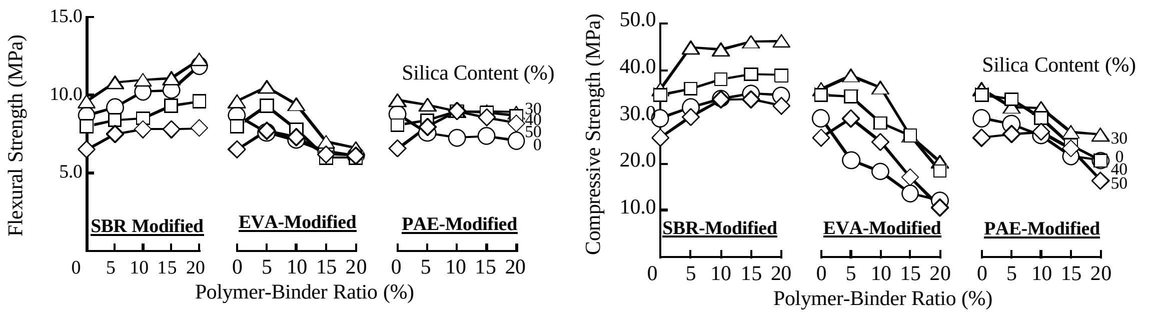


**Figure 4. Polymer-binder ratio vs. flexural and compressive strengths of autoclaved polymer-modified mortars with silica contents of 0, 30, 40 and 50%**

polymer-modified mortars with slag and silica contents of 0, 30, 40 and 50%, respectively. Regardless of the type of polymer and the polymer-binder ratio, except for the flexural strength of EVA- and PAE-modified mortars using slag, the flexural and compressive strengths of the autoclaved polymer-modified mortars using slag and silica tend to increase with increasing slag and silica contents, and to reach maximums at slag and silica contents of 30%. The flexural strength of EVA- and PAE-modified mortars using slag tends to decrease with increasing slag content irrespective of the polymer-binder ratio. In particular, this tendency is significant at larger polymer-binder ratios, and saponification of the polymers is suggested from this tendency. SBR-modified mortars using slag and silica are superior in flexural and compressive strengths to EVA- and PAE-modified mortars using slag and silica. The development of higher strengths is attributed to no degradation of the polymers and the accelerated pozzolanic reaction of slag and silica by autoclaving (Ohama et al. 1990). Regardless of the slag and silica contents, the flexural and compressive strengths of SBR-modified mortars using slag and silica tend to increase with increasing polymer-binder ratio or reach maximums at a polymer-binder ratio of 20%, whereas those of EVA- and PAE-modified mortars using slag and silica tend to decrease with increasing polymer-binder ratio. This may be explained by the conditions of the polymer films formed in the autoclaved polymer-modified mortars as shown in Figs. 5 and 6. In the figures, tough polymer films are observed in SBR-modified mortars, but brittle and torn polymer films due to their saponification by autoclaving are evident in EVA- and PAE-modified mortars.

| SBR-Modified | EVA-Modified | PAE-Modified |
|---|---|---|

**Figure 5. Microstructures of autoclaved polymer-modified mortars with polymer-binder ratio of 20% and slag content of 30%**

| SBR-Modified | EVA-Modified | PAE-Modified |
|---|---|---|

**Figure 6. Microstructures of autoclaved polymer-modified mortars with polymer-binder ratio of 20% and silica content of 30%**

### 4.2 Polymer durability

Table 4 shows the basic physical properties of polymer films. Tables 5 and 6 give the physical properties and appearances of the polymer films before and after autoclaving,

respectively. In comparison with the physical properties of the polymer films, there are no marked differences in the minimum film-forming temperature, glass transition temperature and tensile strength between SBR, EVA and PAE films. However, the elongation of SBR film is almost the same as that of EVA film, but the elongation of PAE film is no more than about one-half of that of SBR and EVA films. In comparison with the physical properties of the polymer films before and after autoclaving (under no immersion in a saturated $Ca(OH)_2$ solution), the respective decreases in the tensile strength and elongation of SBR film are about 20%, but the tensile strength of EVA film decreases 40% or more, and the elongation of PAE film decreases about 60%. In comparison with the physical properties of the polymer films before and after autoclaving under immersion in the saturated $Ca(OH)_2$ solution, the respective decreases in the tensile strength and elongation of SBR film are about 25%, but decreases in the tensile strength and elongation of EVA film are 66% and 81%, respectively, the respective decreases in the tensile strength and elongation of PAE film being 43 to 44%. The significant degradation of EVA and PAE films due to the synergistic effect of autoclaving and saturated $Ca(OH)_2$ solution immersion explains the reduced physical properties of EVA and PAE films compared to SBR films. This corresponds with the changes in appearance of SBR, EVA and PAE films before and after autoclaving under immersion in the saturated $Ca(OH)_2$ solution as seen in Table 6.

**Table 4. Basic physical properties of polymer films.**

| Type of polymer film | SBR film | EVA film | PAE film |
|---|---|---|---|
| Appearance | Transparent film | Transparent film | Transparent film |
| Minimum film-forming temperature (°C) | 5.0 | 0 | 7.0 |
| Glass transition temperature (°C) | -6.0 | -3.6 | 0.1 |
| Tensile strength (MPa) | 6.67 | 4.79 | 5.28 |
| Elongation (%) | 922 | 1037 | 436 |

Figure 7 represents the infrared spectra of SBR films before and after autoclaving under immersion in a saturated $Ca(OH)_2$ solution. There is hardly a difference in the infrared spectrum between SBR films before and after autoclaving under immersion in

the saturated $Ca(OH)_2$ solution. This fact suggests that SBR film is hardly degraded under such autoclaving conditions, i.e., SBR films in SBR-modified mortars are not degraded by autoclaving.

**Table 5. Physical properties of polymer films before and after autoclaving.**

| Type of polymer film | | SBR film | EVA film | PAE film |
|---|---|---|---|---|
| Before autoclaving | Appearance | Transparent film | Transparent film | Transparent film |
| | Tensile strength (MPa) | 6.67 | 4.79 | 5.28 |
| | Elongation (%) | 922 | 1037 | 436 |
| After autoclaving | Appearance | Transparent film with a few small wrinkles | Somewhat yellowed film with some large or small wrinkles | Transparent film with many small wrinkles and cavities |
| | Tensile strength (MPa) | 5.35 | 2.68 | 5.81 |
| | Tensile strength change (%) | -20 | -44 | 10 |
| | Elongation (%) | 766 | 920 | 191 |
| | Elongation change (%) | -17 | -11 | -57 |
| After autoclaving in saturated $Ca(OH)_2$ solution | Appearance | Whitened film with some shallow depressions | Whitened film with many large wrinkles and small cavities | Whitened film with many large wrinkles |
| | Tensile strength (MPa) | 5.05 | 1.64 | 3.48 |
| | Tensile strength change (%) | -24 | -66 | -44 |
| | Elongation (%) | 682 | 194 | 250 |
| | Elongation change (%) | -26 | -81 | -43 |

**Table 6. Polymer films before and after autoclaving.**

| Type of polymer film | Before autoclaving | After autoclaving | After autoclaving in saturated $Ca(OH)_2$ solution |
|---|---|---|---|
| SBR film |  |  |  |
| EVA film |  |  |  |
| PAE film |  |  |  |

Figure 8 illustrates the infrared spectra of EVA films before and after autoclaving under immersion in a saturated $Ca(OH)_2$ solution. In comparison with the infrared spectra of EVA films before and after autoclaving under immersion in the saturated $Ca(OH)_2$ solution, in EVA film after autoclaving, the peak height from hydroxyl group (-OH) at around 3300$cm^{-1}$ increases, and that from ester linkage (-COO-) at 1720 $cm^{-1}$ decreases. This is attributed to the formation of ethylene vinyl alcohol and water-soluble calcium acetate by the partial saponification of EVA polymer with $Ca(OH)_2$(Yasu & Kobayashi 1994) as shown in Fig. 9.

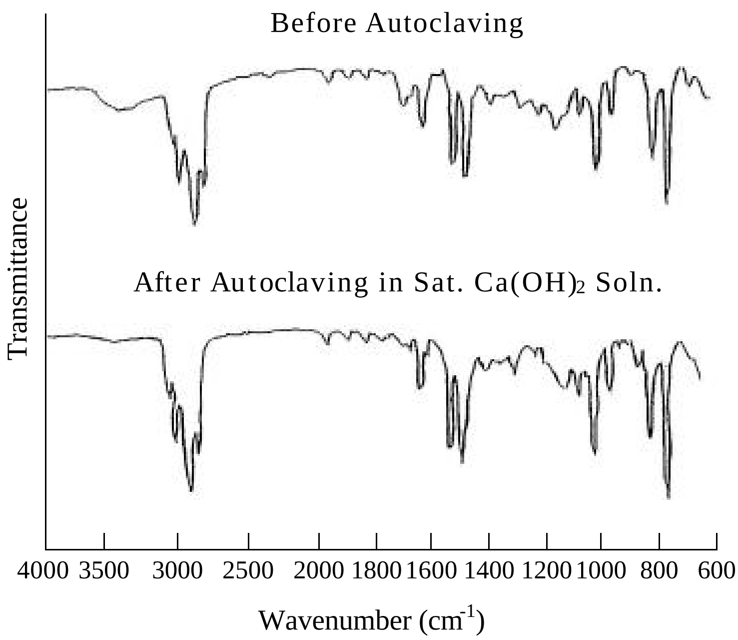


**Figure 7. Infrared spectra of SBR films before and after autoclaving under immersion in saturated $Ca(OH)_2$ solution**

Figure 10 exhibits the infrared spectra of PAE films before and after autoclaving under immersion in a saturated $Ca(OH)_2$ solution. In comparison with the infrared spectra of PAE films before and after autoclaving under immersion in a saturated $Ca(OH)_2$ solution, in PAE film after autoclaving, the peak height from ester linkage (-COO-) at 1720cm$^{-1}$ decreases, and the peak at 1580 cm$^{-1}$ arises from carboxylate . This is due to the formation of carboxylate (calcium polyacrylate) and alcohol by the partial saponification of PAE polymer with $Ca(OH)_2$(Yasu & Kobayashi 1994) as seen in Fig.11.

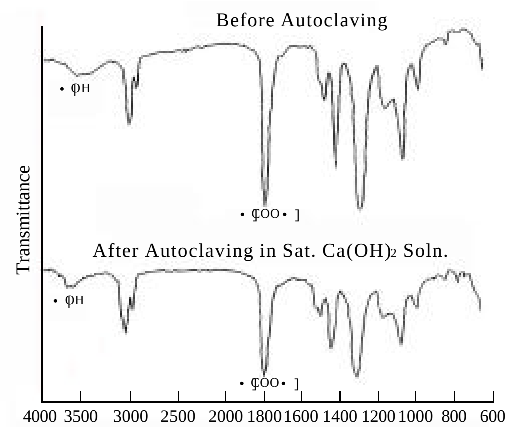


**Figure 8. Infrared spectra of EVA films before and after autoclaving under immersion in saturated $Ca(OH)_2$ solution**

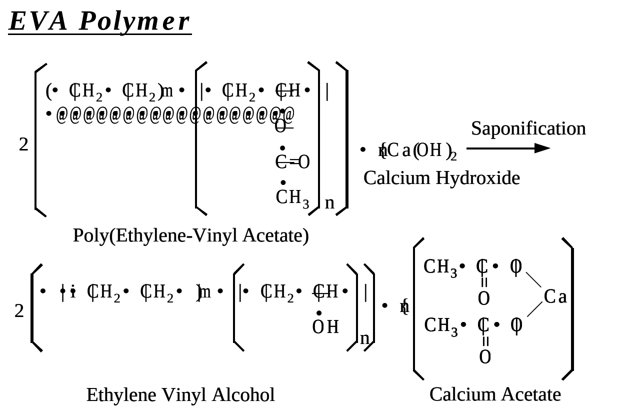


**Figure 9. Mechanism of saponification of EVA polymer**

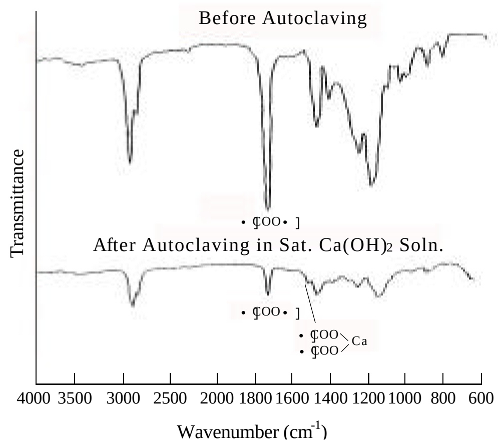


**Figure 10. Infrared spectra of PAE films before and after autoclaving in saturated $Ca(OH)_2$**

*PAE Polymer*

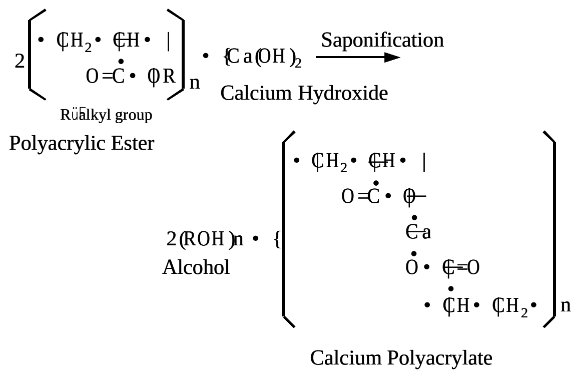


**Figure 11. Mechanism of saponification of PAE polymer**

## 5 CONCLUSIONS

The conclusions from the test results are summarized as follows:

1. Regardless of the type of polymer and polymer-binder ratio, except for the flexural strength of autoclaved EVA- and PAE-modified mortars using slag, the flexural and compressive strengths of autoclaved polymer-modified mortars using slag and silica tend to increase with increasing slag and silica contents, and to reach maximums at slag and silica contents of 30%. Irrespective of the slag and silica contents, the flexural and compressive strengths of autoclaved SBR-modified mortars using slag and silica tend to increase or reach maximums at a polymer-binder ratio of 20% with increasing polymer-binder ratio, whereas those of autoclaved EVA- and PAE-modified mortars using slag and silica tend to decrease with increasing polymer-binder ratio.

2. The application of autoclaving to polymer films in a saturated $Ca(OH)_2$ solution results in around 25% decreases in the tensile strength and elongation of SBR film and 40 to 80% decreases in those of EVA and PAE films.

3. Autoclaving polymer films in a saturated $Ca(OH)_2$ solution results in no significant change in the infrared spectrum for SBR film, however, causes marked changes in the infrared spectrum for EVA and PAE films because of the partial saponification of EVA and PAE polymers. This fact suggests that SBR film is hardly degraded under such autoclaving conditions.

4. From the above results, autoclaved EVA-and PAE-modified mortars using slag and silica appear to be degraded by the saponification of EVA and PAE polymers, but autoclaved SBR-modified mortars using slag and silica are not degraded. Accordingly, autoclave curing can be applied to SBR-modified mortars using slag and silica. In this case, the use of slag rather than silica is recommended.

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# Development Of A Durability Branding System For Steel Construction Products

**JC Robinson**
**Industrial Galvanizers Corporation**

**Summary: There is presently no system available to specifiers in Australia to define durability on coated steel products, in particular, zinc coated and galvanized steel products. The author describes the development of a Durability Branding and Certification system that will allow the durability of metallic steel coatings to be defined using a Star Rating system. The Star Rating Durability Certification is based on protocols defined in existing ISO (International Standards Organisation) standards and will provide a performance-based visual standard to supplement the existing disconnected prescriptive standards.**

**Keywords: Durability, certification, galvanized, specification.**

## 1 INTRODUCTION

Galvanized or zinc and zinc alloy coated products represent one of the largest construction material segments used in Australia, with more than one million tonnes of sheet, wire, tube and structural sections being used annually in Australia.

While the galvanized and zinc based coatings on these products are defined in various Australian, Australian/New Zealand, and ISO standards, these standards are largely prescriptive and define minimum requirements for coating mass without reference to durability.

There is a significant difference between the various galvanized and other zinc based coatings in their physical and metallurgical characteristics, and in particular, their durability.

The differences in these coatings in terms of durability are poorly understood at specifier level, and as a result, many products are selection for critical construction applications that do not deliver the expected maintenance free life.

It is not possible to determine the characteristics of many zinc-based coatings simply by their appearance, as coating durability is a function of coating mass (coating thickness), which can only be determined with specialised equipment.

The Galvanizers Association of Australia (GAA) has initiated a program to identify its members' hot dip galvanized products with a Durability Certification label that will allow specifiers and consumers to obtain durability information on the particular galvanized product.

## 2 ZINC COATING PARAMETERS AFFECTING DURABILITY

The most significant factor affecting zinc-based coating durability is coating mass. This is usually specified in grams per square metre ($g/m^2$). In practice, coating mass is usually converted to a relative coating thickness to allow non-destructive testing of the coating to be done with an appropriate thickness gauge, where the coating thickness is defined in microns (µ).

Other factors affecting coating durability include the method of zinc coating application, the presence of other alloying elements in the coating and the existence of surface treatments applied as an integral step in the coating process. A brief description of the main non-proprietary zinc based coatings and their method of application follows.

### 2.1 Zinc electroplating

Zinc electroplating involved immersing pre-treated components in a zinc plating solution and applying a DC current. Zinc electroplating is widely used as a coating on builders' hardware, fasteners and appliance components.

The coating consists of pure zinc with the coating thickness rarely exceeding 15µ and generally measuring less than 10µ.

The coating produced is bright and smooth. Chromate post treatment on some electroplated products is done to improve durability performance and is characterised by a yellow-brown colour.

### 2.2 Continuous or in-line galvanizing

Sheet, wire and some hollow and open sections are galvanized in a continuous process, where the pre-treated steel passes through a galvanizing bath at speeds up to 180 metres per minute. Control systems in the process closely control the coating thickness within the limits of the process. Coatings exceeding 300 g/m$^2$ are rarely applied in these processes and coatings are more commonly in the 15-25µ range.

The coating consists of largely pure zinc with a thin (less than 5µ) zinc-iron alloy layer at the steel/coating interface.

The coating produces is smooth and uniform and may have a `spangled' appearance that arises from alloying elements in the zinc – specifically lead.

Continuously galvanized coatings are applied to semi-finished products, which are always subject to further processing. This results in areas of the steel in the finished being uncoated because of cutting and punching.

### 2.3 Hot dip galvanizing

Batches of fabricated items are immersed in a bath of molten zinc after pre-treatment for periods ranging from 3-10 minutes, depending of the mass and shape of the articles. This produces a relatively thick coating that will vary in appearance depending on the surface condition of the steel, its chemistry, its shape and other design elements.

The coatings consist largely of zinc-iron alloys (75%+) with a surface coating of zinc. The zinc-iron alloys are hard and relatively brittle. This metallurgical characteristic results in the significant differences in abrasion resistance and flexibility when compared to continuously galvanized coatings.

Hot dip galvanized coatings are applied to steel items after fabrication and thus all surfaces are coated. Coating thickness variation will occur as a result of the orientation of the work to the galvanizing bath that affects drainage characteristics. The hot dip galvanized coating can be variable in thickness due to local variations in the surface chemistry of the steel.

Hot dip galvanized coatings range from around 40µ in thickness on thin (less than 3 mm) steel sections to over 200µ on heavy structural sections.

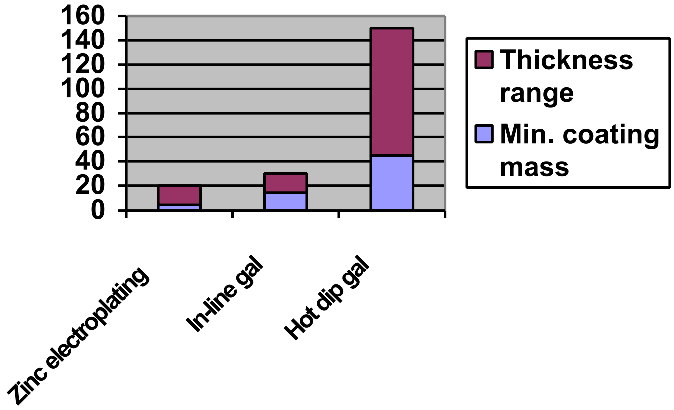


**Figure 1. Zinc (galvanized) coating thickness applied by different methods**

## 3 COATING LIFE AND COATING THICKNESS

Galvanized (zinc) coatings behave differently to applied organic coatings in that their life is determined by the rate at which they oxidise from the surface. In atmospheric exposures, the rate of corrosion is approximately linear and is determined by the classification of the environment.

Thus, the coating thickness is the most important element in determining the maintenance free life of a zinc-based coating. Other secondary, but still important factors determine corrosion rates of zinc (galvanized) coatings. These include orientation of the surface, degree of sheltering. Recent CSIRO test programs have indicated that the presence of significant zinc-iron alloy layers, absent in continuously galvanized products, and always present in hot dip galvanized products may have significantly lower (2X-3X) rates of corrosion that zinc coatings.

This phenomenon provides a reliable method of determining the expected life of a zinc-based coating in any nominated environment, as long as the coating thickness can be determined.

There is a large body of corrosion data collected over 100 years on the corrosion rate of zinc. In more recent times, computer modelling of surface oxidation reactions has been added to the empirical data to further refine corrosion rate data for a wide range of actual and theoretical environments.

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The International Standards Organisation (ISO) has developed a suite of standards to quantify zinc corrosion rates and classify environments to which they are exposed.

The key ISO standards are:

ISO 9223 – *Corrosion of metals and alloys – Corrosivity of atmospheres – Classification*

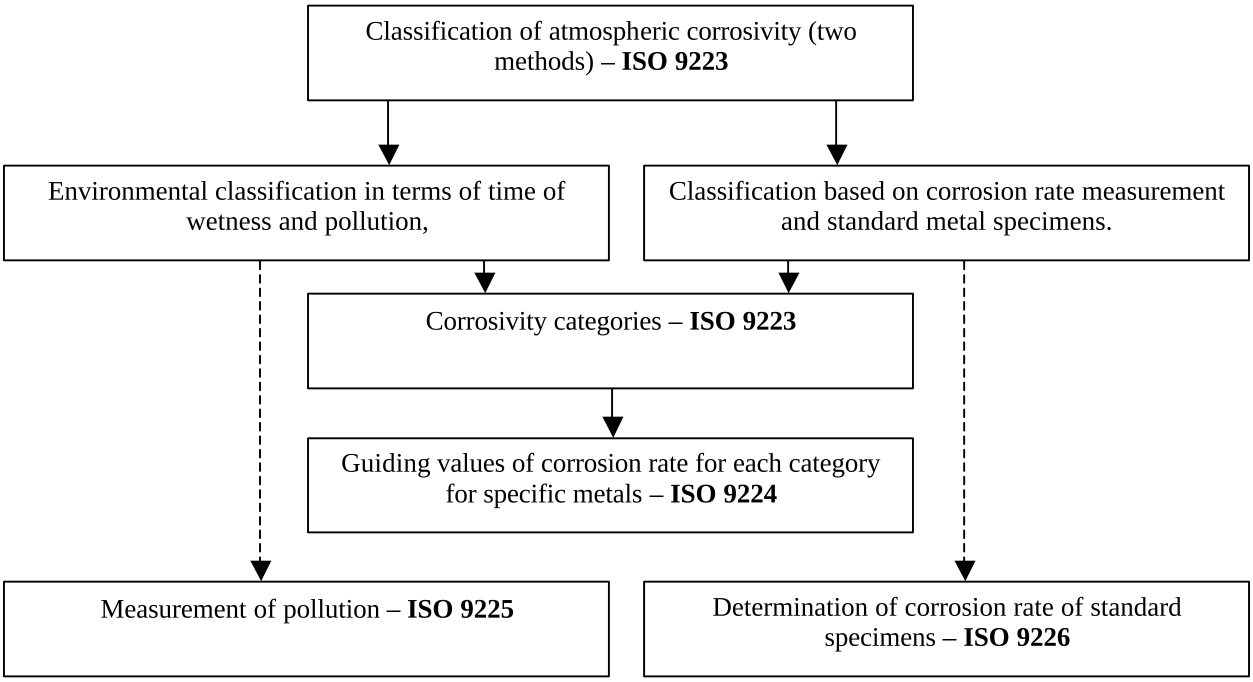
ISO 9223 – *Corrosion of metals and alloys* - *Corrosivity of atmospheres – Guiding values for the corrosivity categories*

ISO 9225 – *Corrosion of metals and alloys – Corrosivity of atmospheres* - *Measurement of Pollution*

ISO 9226 - *Corrosion of metals and alloys - Corrosivity of atmospheres- Determination of corrosion rate of standard specimens.*

These standards take a structured approach to classifying atmospheres using time of wetness, temperature, chloride and $SO_2$ levels.

**Table 1. Classification of atmospheric corrosivity**




These ISO standards use a C1-C5 classification to define atmospheric corrosion classification. These categories are defined as follows:

**Table 2. Categories of corrosivity of the atmosphere**

| **Category** | **Corrosivity** |
|---|---|
| C1 | Very low |
| C2 | Low |
| C3 | Medium |
| C4 | High |
| C5 | Very high |

These ISO categories are broad and non-descriptive and need to be related to practical environments to which they are applied. AS/NZS 2312 has used ISO 9223 to produce a set of atmospheric classifications more appropriately descriptive of regional environmental exposures.

These are:

1. *Interior exposure (ISO Category C1)* - Steel corrosion rates less than 1.3µ/year.
2. *Mild (ISO Category C2)* - Steel corrosion rates up to 10µ/year. Most rural environments are in this classification.

3. *Moderate (ISO Category C2)* - Steel corrosion rate from 10-20µ/year. Most capital city and suburban areas are in this classification.
4. *Tropical (ISO Category C2)* – In Australia, this includes coastal areas north of the Tropic of Capricorn. This is a category that cannot be readily delineated by ISO 9223 parameters. Measurements put this into an ISO C2 category for metals but it is classed as more aggressive for organic coatings.
5. *Industrial (ISO Category 3-4)* – First year steel corrosion rates greater than 25µ/year, but may extend into ISO Category 4 (50µ/year). Significant reductions in industrial pollutant levels through regulation has resulted in Australian heavy industrial centres being in the lower range of corrosivity classification.
6. *Marine (ISO Category 3)* – First year corrosion rates of 25-50µ/year, and within 1 km of the ocean. Topography and climatic conditions will influence the transport of chlorides, which are the major determining factor.
7. *Severe marine (ISO Category 4-5)* – First year steel corrosion rates exceeding 50µ/year includes off-shore and coastal areas subject to ocean surf and prevailing on-shore winds. Most of the Australian oceanfront with exception of those areas sheltered by reefs and islands are represented in this category.

While there is relativity in the corrosion rate of zinc and steel in a given corrosivity classification, it is not constant across all corrosivity classifications.

Table 3 lists typical corrosion rate ranges in the above AS/NZS 2312 atmospheric classifications.

**Table 3 – Typical corrosion rate range – Zinc and steel**

| **Corrosivity classification** | **Corrosion rate zinc - m/year** | **Corrosion rate steel - m/year** | **Zinc/steel corrosion ratio (approx)** |
|---|---|---|---|
| Mild | <1 µ/year | <10 µ/year | 1:10 |
| Moderate | <2 µ/year | 10-20 µ/year | 1:20 |
| Tropical | < 2 µ/year | 20-50 µ/year | 1:50 |
| Industrial | 2-4 µ/year | 20-50 µ/year | 1:15 |
| Marine | 2-4 µ/year | 20-80 µ/year | 1:20 |
| Severe marine | 4->10 µ/year | 80-200 µ/year | !:20 |

By comparing this corrosion rate information with the measured or minimum specified zinc (galvanized) coating thickness, an accurate assessment of expected coating life can be made. e.g. An 84µ thick (600 g/m$^2$) hot dip galvanized coating in a *Moderate* corrosivity classification would have an expected coating life to first maintenance of 40+ years.

Other documents and standards, such as AS 3700 – *Masonry structures* use other nomclementure to define corrosivity classifications but these can be generally cross-referenced with the ISO protocols.

## 4 EXISTING BUILDING CODES AND STANDARDS

There is a large number of standards related to the application and constitution of steel coatings of all kinds, along with their pre-treatments. AS/NZS 2312:1994 *Guide to the protection of iron and steel against atmospheric corrosion*, references over 50 standards for various types of galvanizing, priming, paint, surface preparation and testing.

These standards are then referenced in other documents such as the Building Code of Australia (BCA) produced by the Building Code of Australia Board (BCAB), which is a federal government body convened to standardise state, territory, local government and building industry codes and practices.

Other organisations such as (CIS) Construction Information Systems (previously NATSPEC) provide specification services by pre-packaging standards and specifications into project related documents to facilitate the documentation and administration of project specifications.

The main focus of the BCA is to document engineering and safety standards for dwellings. It largely references other standards for durability information, and in some areas, such as shelf angles and lintels, contradictions exist within the referenced standards.

The main weakness with the BCA is that there are no clearly defined durability standards (e.g. that an element in a dwelling must have a design life of X years.)

The onus of performance thus frequently resides with the builder, and without enforcement, inadequately protected steel elements are used in many jurisdictions.

This is not necessarily because individuals wish to take short cuts, but because of the poor understanding of the differences in performance between metallic coatings on steel building products.

It is difficult, without the necessary measuring equipment or experience, to tell the difference between many zinc (galvanized) coatings and a the durability of a similar looking steel section may be reduced by 5X due to the different technology that has applied the zinc (galvanized) coating.

## 5 STANDARDS RELATED TO VARIOUS ZINC-COATED PRODUCTS.

A suite of Australian (and NZ) standards covers the range of zinc and galvanized coatings that are applied to most building and construction products. Additional standards also cover coatings on specific products such as fasteners, which are not referenced here. These Standards are:

**AS/NZS 4680 -** Hot dip galvanized (zinc) coatings on fabricated ferrous articles**AS/NZS 4534 -** Zinc and zinc/aluminium coatings on steel wire
**AS/NZS 4791 -** Hot dip galvanized (zinc) coatings on ferrous open sections applied by a continuous or specialised process.
**AS/NZS 4792 -** Hot dip galvanized (zinc) coatings on ferrous hollow sections applied by a continuous or specialised process.
**AS 1397 -** Steel sheet and strip – Hot dipped zinc coated and aluminium/zinc coated coated
**AS1789 -** Electroplated coatings – Zinc on iron and steel
**AS 4750 (Int.)** – Electro-galvanized (zinc) coatings on ferrous hollow sections.

Each of these standards defines coating mass/coating thickness requirements and the method of designation, using a coating descriptor and a number representing the mass per $m^2$ of the coating.

For example, Z is used for zinc (galvanized), ZA for zinc-aluminium alloy, E for electroplated and ILG for in-line galvanised tube and open sections. The numeral (150, 300, etc) is the coating mass in $g/m^2$.

All standards with the exception of AS 1397 specify the single-side coating mass. AS 1397 specifies the total coating mass on both sides of the galvanized sheet. Thus a Z350 class coating on galvanized sheet is equivalent to a Z175 class on other galvanized products. This, in itself, is a constant source of confusion to specifiers attempting to define durability.

## 6 DURABILITY BRANDING OF ZINC COATED (GALVANIZED) STEEL PRODUCTS

The need to classify the performance of buildings and building components is well established. Local, state and federal authorities have implemented performance based `branding’ protocols to determine energy efficiency of buildings and the engineering and energy ratings of building elements such as windows.

`Star’ rating systems are commonly used on items and services as disparate as appliances and accommodation to certify the performance of the `product’ in a way that is easily recognised by the consumer.

The Galvanizers Association of Australia (GAA) has long been faced with the problem of differentiating the high performance, heavy-duty hot dip galvanized coatings applied by its members and the many other coatings that claim the same credentials as a `galvanized’ coating.

While the characteristics of many of these many coatings are clearly defined in the Standards relating to these coatings, these differences are sometimes reluctantly revealed to the marketplace and at specifier level. There is little knowledge of the specific Australian standards related to these types of products or their relative performance.

The first stage of the national Durability Branding Program being implemented by the GAA involves the certification of its members’ hot dip galvanized products with a labelling system that will allow specifiers to easily identify coating durability with an identifier attached to the product (sticker, label, tag).

Because the corrosion rate of galvanized coatings is essentially linear, and the coating thickness is clearly defined by the process and is relative to steel section thickness and process metallurgy, a clear definition of galvanized coating durability can be provided on products hot dip galvanized after fabrication.

GAA member galvanizers have accredited coating measuring equipment to accurately determine coating thickness on any steel section processed.

By allocating one `Star’ for each 25 microns of galvanized coating thickness, a rating system covering the full range of hot dip galvanized coatings can be provided. The `Stars’ are then related to an atmosphere corrosivity classification table to give an immediate determination of coating life expectancy in any environmental classification.

A sample of a GAA Durability Branding Certificate is illustrated in *Figure 2*

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galvanizers
ASSOCIATION OF AUSTRALIA

**Durability Certification**

This coating complies with AS/NZS 4680:1999. The Durability Certification is derived from ISO 9223 and indicated galvanized coating life expectancy is shown in the table.

For verification of environmental corrosivity classification, refer to AS/NZS 2312 in conjunction with ISO Standards 9223,9224,9225 and 92266.

☆☆☆☆☆☆

Expected coating life

| Star rating | Mild | Moderate | Industrial | Marine |
|---|---|---|---|---|
| ☆ | 10 | 7 | 5 | 3 |
| ☆☆ | 20 | 15 | 10 | 5 |
| ☆☆☆ | 35 | 25 | 15 | 8 |
| ☆☆☆☆ | 50 | 30 | 20 | 10 |
| ☆☆☆☆☆ | 60 | 40 | 25 | 12 |
| ☆☆☆☆☆☆ | 75 | 50 | 30 | 15 |

GAA Member Certification Number XX12345

Certified coating thickness Microns

135m

**Figure 2. Sample GAA Durability Branding Certificate**

This labelling system provides a simple and comprehensive identifier for specifiers seeking to have galvanized coatings supplied to a performance standard.

## 7 COATING CERTIFICATION

Where independent certification of the coating is required, this is available through organisations such as CSIRO or BRANZ, who are already actively involved in product and coating evaluation and also support the development of performance-based standards as a policy.

BRANZ (Building Research Association of New Zealand) has considerable regulatory authority to enforce durability-based standards in New Zealand. A minimum building design life of 50 years for structural elements and lower specified life for more easily replaced components.

The protocols established by BRANZ in New Zealand could well be duplicated by Australia's BCA in providing a regulatory framework with more enforceable elements that the present BCA.

The long-term aim of the Durability Branding initiative is to expand the program to include a full range of coated steel products used for building and construction, including fasteners, coated cladding and roofing, and process controlled paint coatings such as powder coating.

Because of the nature of these various coatings, independent methods of durability certification will need to be developed to allow them to durability certified. Both CSIRO, BRANZ and other testing organisations have developed accelerated durability testing procedures that may satisfy the requirements for defining long-term durability performance.

Certification of zinc coated (galvanized) of products outside the jurisdiction of the GAA can also be done through one the above certifying agencies using the same performance parameters as are proposed by the GAA.

This will allow a large range of existing steel building products to be `Durability Certified', by having them submitted for certification to the certifying agency by their supplier/manufacturer. This may not be a welcome initiative with many existing steel building products, in particular builders' hardware and some in-line galvanized light structural and hollow sections, is the relatively poor performance of the zinc coatings on these products in outdoor exposures will be immediately evident to the consumer.

Many of these `galvanized' products would not rate one ☆ using the GAA Durability Certification and this will ensure that the higher durability products get due consideration in the selection process, where they are currently disadvantaged because of the low cost of those coatings with less durable zinc (galvanized) coatings.

## 8 CORROSION MAPPING DEVELOPMENTS FOR DURABILITY MANAGEMENT

In 2001, Industrial Galvanizers Corporation and the CSIRO entered into a research agreement to develop an Internet based Corrosion Mapping System (CMS) to provide on-line environmental corrosion data for a range of materials, but specifically steel and galvanized coatings.

The CMS will allow specifiers to obtain corrosion rate data in any location in Australia and relate this to durability, given section or coating thickness of the material being evaluated.

While this has been developed independently by Industrial Galvanizers, the synergy with the Durability Branding program is obvious, as it not only allows the Durability Certification provided through GAA members and other participants to be verified independently (all the corrosion data comes from the CSIRO's database) but also allows durability assessments to be made on existing galvanized steel products.

The CMS is expected to be fully operational by January 2002 and a demonstration version is available on the *Technical* page at www.indgalv.com.au.

## 9 SUMMARY

It is not possible to accurately determine the durability of structures unless the components used in their construction are able to satisfy the durability requirements of the design. Without a method of certifying the durability of coated steel products, the selection of materials becomes subjective and is often dictated by expediency and cost rather than performance.

Premature corrosion of steel building components is one of the major causes of degradation and impacts on the life-cycle costs of the building, its future value and its safety.

Support of this GAA Durability Branding initiative by the specifying community, and by the administrators of building codes at national, state and local government level will ensure; that optimum material performance is incorporated into buildings; that environmental impacts caused by corrosion are eliminated; that maintenance costs are minimised and that a technically sound basis for determining durability is available at the 'point of sale' using Durability Certification labelling.

The development of broader-based Durability Branding protocols for other coated steel building products will ensure that better control of building durability can be implemented from the design stage through to construction.

The long-term strategy is to facilitate the development of Australian standards and building codes that incorporate performance-based durability requirements, and ultimately extend this to ISO standards so that international practices are consistent in ensuring the durability of coated steel construction materials.

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# Biofilms On Porous Building Materials: Friend Or Foe?

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**Summary: Current models generally view biofilms as having a detrimental role in the weathering process of building materials. The work covered by this paper will highlight that this may not be the complete story.**

**The degradation of many building materials is directly linked to the occurrence and movement of fluids within their porous structure, leading to secondary problems such as freeze-thaw damage, salt shattering, clay swelling, mineral leaching and precipitation. It is also thought that biofilm development on porous materials compounds this problem by absorbing water and acting like a wick in the transport of fluids from the external environment into the matrix of the building material.**

**However, experimental results indicate that biofilms can demonstrate hydrophobic properties, repelling water and consequently inhibiting the passage of water into the porous network of building materials. The water phase wettability of quartz was used to assess this characteristic.**

**The initial wettability of quartz fragments was determined using an environmental scanning electron microscope (ESEM). This showed the quartz fragments to be hydrophilic. The quartz fragments were then inoculated with microbes and incubated, facilitating biofilm development on their surfaces. The quartz fragments were re-examined using the ESEM, showing that quartz fragments coated in biofilm displayed hydrophobic behaviour. Hence, the addition of a biofilm to quartz can change the surface characteristics from hydrophilic to hydrophobic.**

**As quartz is the major component of sandstone this may help to understand the durability of sandstone undergoing water infiltration. Since biofilm development is usually associated with areas of building facades that are prone to water logging, the presence of the biofilm itself need not necessarily be a contributing factor to the problem. *In situ* biofilm may in fact impede ingression of external water sources into porous building materials by behaving as a water resistant layer. On the other hand the presence of a hydrophobic biofilm layer may add to the problem of water retention within porous building materials. Regardless of whether fluids are prevented from ingressing or egressing by hydrophobic biofilm barriers, either will have profound implications for the durability of porous building materials.**

**KEYWORDS: biofilm, wettability, sandstone, weathering, protection.**

## 1 INTRODUCTION

Previous research has shown that building materials, such as sandstone, limestone, granite, concrete and mortar, which are used in diverse climates, are readily colonised by microbes when the environment for growth is favourable. The microbes that have been studied include bacteria, cyanobacteria, fungi, lichens, and to a lesser extent algae (Paine *et al.* 1933, Milde *et al.* 1983, Palmer & Hirsch 1991, Urzi *et al.* 1991, Zherebyateva *et al.* 1991, Adams *et al*. 1992, De La Torre *et al.* 1993, Urrutia & Beveridge 1995, Jozsa *et al.* 1996, Wilimzig & Bock 1996, Ferris & Lowson 1997, Flores *et al.* 1997, Urzi & Realini 1998, Uchida *et al*. 1999).

Microbial-rock interactions are of considerable interest as microbes are thought to have a significant role in the degradation of building materials (Lyalikova & Petushkova 1991, Bock & Sand 1993, Hirsch *et al.* 1995a, b, Viles 1995, Barker & Banfield 1996, Palmer & Hirsch 1996, Feldman *et al.* 1997, Barker *et al.* 1998, Rogers *et al.* 1998, Papida *et al.* 2000). The problems associated with microbial attack include chemical and mechanical weathering. Chemical mechanisms include: dissolution, precipitation, and alteration of mineral phases (Henderson & Duff 1963, Weed *et al.* 1969, Huang & Keller 1970, Lauwers & Heinen 1974, Halsey *et al.* 1995, Paris *et al.* 1995a, b, Spirakis 1996, Barker & Banfield 1998, Ehrlich 1998, Sweevers *et al.* 1998, Chigira & Oyama 1999; Labrenz *et al.* 2000, Liermann *et al.* 2000). Mechanical mechanisms include salt shattering, mineral expansion, freeze-thaw shattering, hydration-dehydration cycling and microboring (De La Torre *et al.* 1993, Halsey *et al.* 1995, Flores *et al.* 1997, Grondona *et al.* 1997, Wright 2000).

Microbes are also strongly associated with areas of high moisture retention such as areas of poor drainage. Once colonised in such areas they are thought to contribute to the problems associated with retention and or movement of moisture in the building material (Riley & Heiman 1996). However it has been demonstrated that biofilms can be a protective agent to building materials. An example of this was highlighted by Grondona *et al.* (1997) in their research surrounding the Casa Lis, a building in Salamanca in Spain which was built with Villamayor sandstone. In this instance, during restoration it was found that the sandstone that had developed an outer biofilm layer had suffered no underlying decay, whereas the sandstone that had no biofilm development was so severely altered that new stone was required to repair the damage. The preserved sandstone was apparently protected from salt crystal formation within the pores. Salt crystal formation within the pore space can cause salt shattering of the surrounding matrix. It is thought that the biofilm prevented evaporation of the fluid within the pore space, thus inducing a stable humidity that inhibited salt crystal formation. While the mechanisms at play in the biofilm are not fully understood, there is a high probability that the wettability characteristics may have had a significant role in preserving the sandstone.

The water phase wettability of a biofilm is an important factor effecting water placement and movement within porous building materials. Wettability describes the relative affinity of a fluid for a substrate (Walker 1999), and determines whether a fluid such as water will spread out across a substrate or be repelled and form discrete droplets. The degree of wettability is commonly measured by the contact angle that a droplet of fluid forms on a solid (Bascom 1992, Schoff 1992, Xie & Morrow 1998, Liu & Buckley 1999). In the case of water, when measured through the densest fluid phase, in this case the water itself, a low contact angle will be observed where the substrate is hydrophilic (Fig.1a), the water droplets form low domes, or occur as discrete sheets. Whereas, a high contact angle will occur on hydrophobic surfaces (Fig.1c), where water will occur in the form of discrete spherical droplets. Where a substrate is hydrophilic, it can be said to be water wet, which, under ambient conditions, is the case for most sandstones composed of predominantly quartz and feldspar (Barclay & Worden 2000). Whereas those that are hydrophobic, such as some calcitic limestones are regarded as oeleophilic, or oil wet (Barclay & Worden 2000). The wettability of a biofilm will therefore affect the relationship between water and a substrate, either on, or within the porous building stone. Wettability will influence the biofilms ability to retain water, as well as affecting the combined biofilm and rock substrate's permeability, and thereby exert a control upon the location and movement of water within the biofilm-porous building material system. As many potentially damaging processes such as leaching, salt shattering and freeze-thaw shattering depend upon the presence of water, the relative water wettability of a biofilm will have major implications for the durability of porous building materials.

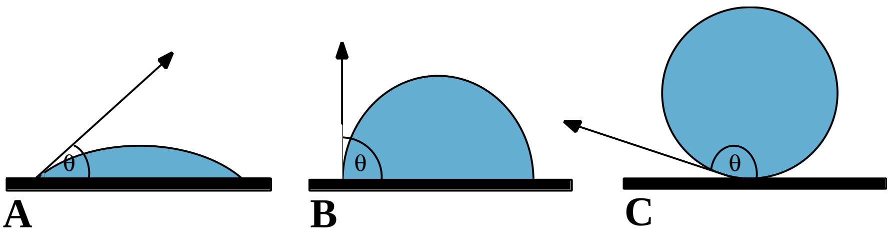


**Figure 1. Contact angles of water droplets, measured in a water vapour atmosphere, on a solid surface (e.g. quartz and biofilm). A) Low contact angle (< 90°), forming on an hydrophilic surface. B) Intermediate contact angle (~90°), forming on a surface of intermediate wetness (neither hydrophilic nor hydrophobic). C) High contact angle (>90°), forming on an hydrophobic surface.**

The wettability of any building material is a controlling factor upon the movement of water on its surface or within its pore spaces. The work highlighted in this paper will show that biofilms that grow on building materials can in fact alter the original wettability of that building material. In some instances this could be favourable and in others less so. The organisms studied include a mixed bacterial-fungal biofilm, an algal biofilm, and a lichen, all of which demonstrated different wettabilities.

## 2 Materials and Methods

### 2.1 Sample preparation

An analytical grade quartz crystal was placed on an oil free sterile surface and crushed with a sterile hammer. Thirty small chips approximately 2-3 mm in length were selected from the debris and stored in individual sterile Petri dishes. The chips were taken from the inner region of the crystal to ensure that only fresh, uncontaminated material was utilised. The quartz chips were handled at all times using sterile forceps to avoid contamination.

### 2.2 Initial wettability

Using a Peltier stage equipped Phillips XL30 ESEM with $LAB_6$ gun, and a 500 micron aperture Gaseous Secondary Electron Detector (GSED), the initial water phase wettability of each quartz chip was established.

The chips were pre-cooled in a standard refrigerator, placed on the Peltier stage in the ESEM chamber and maintained at 5°C. Wet mode was selected from the ESEM controller, and the chamber was pumped to 5 Torr. Upon attaining 5 Torr, the chamber was passed through up to five 'flooding' cycles from 5 to 10 Torr. Flooding with water vapour provides a suitable atmosphere for charge suppression, facilitates image amplification, maintains sample hydration, and most importantly to this experiment, supplies a wetting medium, water.

Chamber pressure was then increased to 6.5 Torr. At a pressure of 6.5 Torr and a temperature of 5°C, relative humidity reached 100%, and water condensed on to the surface of the sample. Observations of water droplet morphology (contact angle) were then recorded.

Where necessary (i.e. if flooding occurred), the whole process was repeated by pumping the chamber to 3 Torr, effectively sublimating all surface water, and returning to 6.5 Torr to repeat the procedure.

Images were acquired at a temperature of 5°C, a pressure of 6.5 Torr, a working distance of approximately 7.5 mm, an operating voltage of 20 kV, and a spot size of between 4 and 6.

Elemental analysis was carried out on each sample to ensure that the quartz chips used were completely pure. An Energy Dispersive X-ray Detector (EDX) was used at pressure 5 Torr and working distance 10 mm. X-rays were collected for 100 live seconds, with a count rate of approximately 1500 and a dead time of approximately 30%.

### 2.3 Biofilm development

In order to coat the quartz fragments with biofilm, a suitable inoculum was prepared. One 0.1 ml loop of each microbial culture was taken from 8 pure isolates of bacteria and one pure isolate of fungi. The loops of microbes were transferred into a conical flask containing 250 ml of Medium X (a growth medium composed of 50% nutrient broth and 50% malt extract broth). The microbial mixture was incubated at 25°C for 2 days to facilitate the development of a microbial consortium.

Using sterile forceps, 20 quartz fragments were individually transferred into twenty 100 ml conical flasks. Ten of the conical flasks contained 25 ml of distilled water and 10 ml of Medium X. Ten of the conical flasks contained 25 ml of distilled water. The flasks were sealed and sterilised in an autoclave for 15 minutes at 121°C. Once sterilisation was complete and the flasks cooled to room temperature 10 ml of the bacterial-fungal inoculum was added to the 10 flasks containing distilled water. This was carried out using strict aseptic technique. The remaining 10 flasks of distilled water and medium were left intact to provide a control, allowing any effects caused by the water and medium to be observed. All 20 flasks were incubated as above for 2 days; encouraging biofilm development on the 10 inoculated quartz chips.

Following the incubation period the contents of each flask were emptied individually into separate sterile Petri dishes, allowing the quartz chips to be easily isolated.

For the algal biofilm analysis, the above process was repeated using 10 quartz chips, 5 for biofilm development and 5 for controls. The biofilm in this instance was composed of 3 axenic green algal cultures. The broth medium used was Euglena Gracilis Medium (EG) and Jaworski's Medium (JM) in a 1:1 ratio. The biofilm was cultured over 4 days at 16°C in an incubator set at a 16hr light / 8hr dark cycle.

For the lichen analysis a single sample of lichen was removed from a sandstone building and examined. The sample was not cultured.

### 2.4 Secondary wettability

The inoculated quartz chips and the control quartz chips were checked for the development of biofilm, and changes in wettability, using the same method of assessment as initial wettability. The results were imaged and stored using exactly the same method as for initial wettability. The lichen was examined under the same conditions as the biofilms.

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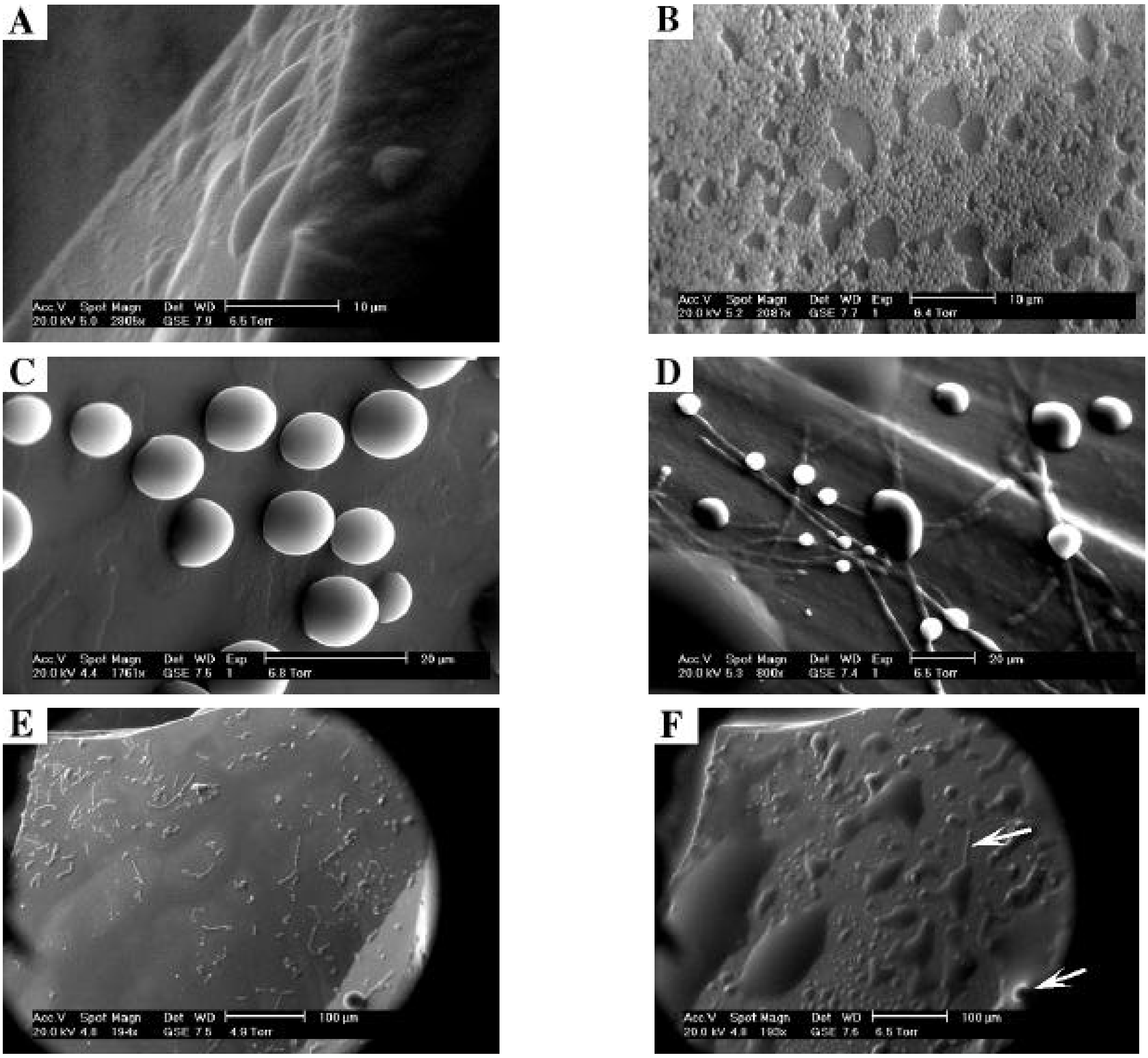


**Figure 2. A) Typical wettability of natural quartz surface, showing low domed hydrophilic droplets. B) Bacterial biofilm on quartz . C) High sphericity hydrophobic droplets on bacterial biofilm grown on quartz. D) High sphericity droplets and high domes, on hydrophobic bacterial-fungal biofilm. E) Quartz surface with algal biofilm. F) The same surface as in (E), displaying hydrophilic low domed drops of water. Two areas with algal cells indicated with arrows.**

## 3 RESULTS

The results of the initial wettability assessment showed that all 30 quartz chips displayed hydrophilic properties and hence their wettability could be described as water wet. Water wetting was typified by low dome droplets displaying contact angles of considerably less than 90°, (Fig. 2a).

The results of the secondary wettability assessment showed that the 10 quartz chips exposed to the bacterial-fungal consortium successfully developed a biofilm (Fig. 2b). The wettability of the quartz chips coated in this biofilm changed from hydrophilic to hydrophobic, typified by high dome droplets with contact angles of > 90°, (Figs 2c, d).

The 5 quartz chips exposed to the algal consortium successfully developed biofilms (Fig. 2e). The wettability of the quartz chips coated in algal biofilm remained hydrophilic, indicated by the low dome droplets of water on the algal surface (Fig. 2f).

The 15 quartz chips used as controls (i.e. those not exposed to microbes), retained their initial hydrophilic wettability (Figs 3a, b).

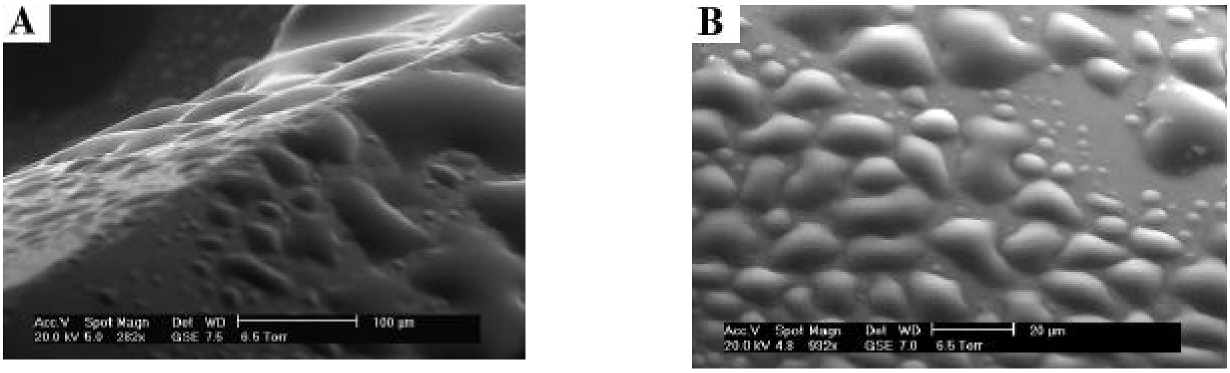


**Figure 3. Control samples: A) Quartz control sample from the bacterial-fungal experiment; B) quartz control from the algal experiment. Note both display low domed droplets of water and are hence still hydrophilic**

The EDX analysis confirmed that the quartz samples used were in fact pure $SiO_2$ and the results were not due to inorganic contamination.

The lichen displayed mixed wettability (Fig. 4b).

## 4 DISCUSSION

The initial wettability of the 30 quartz chips was hydrophilic. This was the expected result, as quartz is widely recognised as such at ambient conditions (Barclay & Worden 2000). The secondary wettability of the 15 control chips remained unchanged, indicating that the water + Medium X broth and the water + EG:JM broth had no impact on the wettability of the quartz chips. Hence, it can be clearly stated that the secondary wettability changes in the inoculated samples were due to biofilm development and no other causes.

Of the 10 samples that underwent bacterial-fungal biofilm development, the wettability altered from hydrophilic to hydrophobic. This was most likely due to an interaction between the water and the bacterial-fungal cellular or extracellular material. The actual biomatter responsible and the mechanisms involved were not investigated, however, it is well known that hydrophobin, a fungal protein, has hydrophobic properties (Wosten *et al.* 1995).

Of the 5 samples that had algal biofilm development, the wettability remained hydrophilic. This indicated that the biomatter involved yielded a similar reaction to the water phase as the quartz.

The sample of lichen that was analysed showed a mixed wettability profile. The portion of the lichen that appeared to be fungal in nature demonstrated hydrophobic tendencies, the portion that appeared to be algal demonstrated hydrophilic tendencies. Given the nature of the biofilm wettabilities outlined above, this was not entirely unexpected.

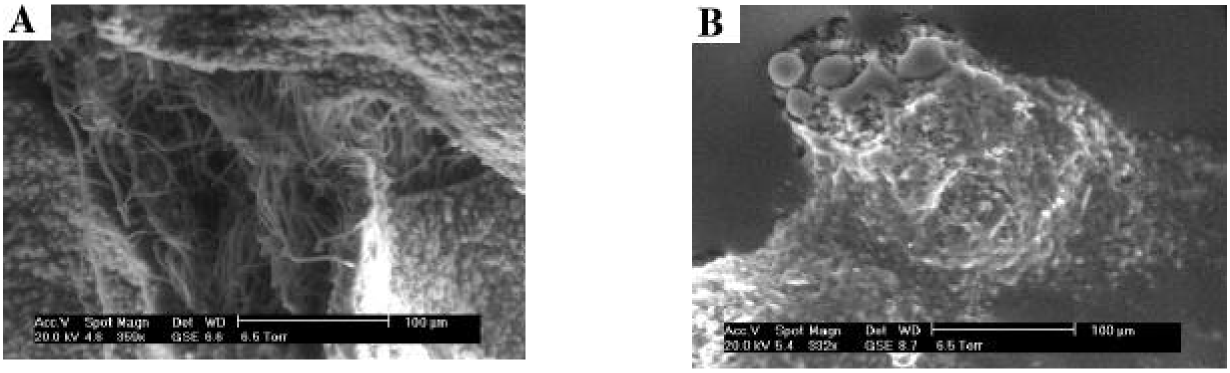


**Figure 4. A) Image of Lichen showing typical morphology, of 'fungal' hyphae and 'algal' cells. B) Typical variation in wettability across lichen. 'Fungal' hyphae (top left) are hydrophobic, indicated by spherical droplets of water, while 'algal' areas (bottom right) have sheets of water on them, typical of hydrophilic substrates.**

It appears to be quite clear that there is no general pattern of wetting characteristics that can be applied to microbes. The characteristics vary from kingdom to kingdom, and quite probably from species to species, therefore no broad assumptions can be made concerning the detailed relationships and interactions between biofilm and water. A biofilm may be predominantly hydrophilic, hydrophobic, or comprise a complex mixture of both.

The consortiums used in this research are quite artificial. In nature biofilms are complex three-dimensional networks comprising many different species of algae, bacteria, fungi and lichens (Bock & Sand 1993, Palmer & Hirsch 1996, Flores *et al*. 1997, Grondona *et al*. 1997). As this is so, they will therefore posses highly varied wettability characteristics, which will have many potential affects on the durability of porous building stone materials. A number of possible scenarios are briefly outlined below:

Hydrophobic biofilms, such as those produced by the bacteria and fungi examined during the present study, are likely to have an important role in controlling water migration through porous building materials. As neither bacteria nor fungi are restricted to the photic zone, they are at liberty to infiltrate deeply into building materials. Previous studies have indicated that bacteria in the presence of water can penetrate several centimetres into sandstone cores (Myers & McCready 1966, Myers & Samiroden 1967, Mills 1997). It is therefore feasible for such hydrophobic biofilms to develop over a much wider range of environments than algal biofilms.

Where an hydrophobic bacterial biofilm develops on the outer surface of a building stone, it could potentially act as a natural waterproofing to the building, possibly preventing ingress of water (Fig. 5a). This could reduce freeze thaw damage, salt shattering, and mineral dissolution and could leave the building stone in a less damaged condition than one without a biofilm *in situ*.

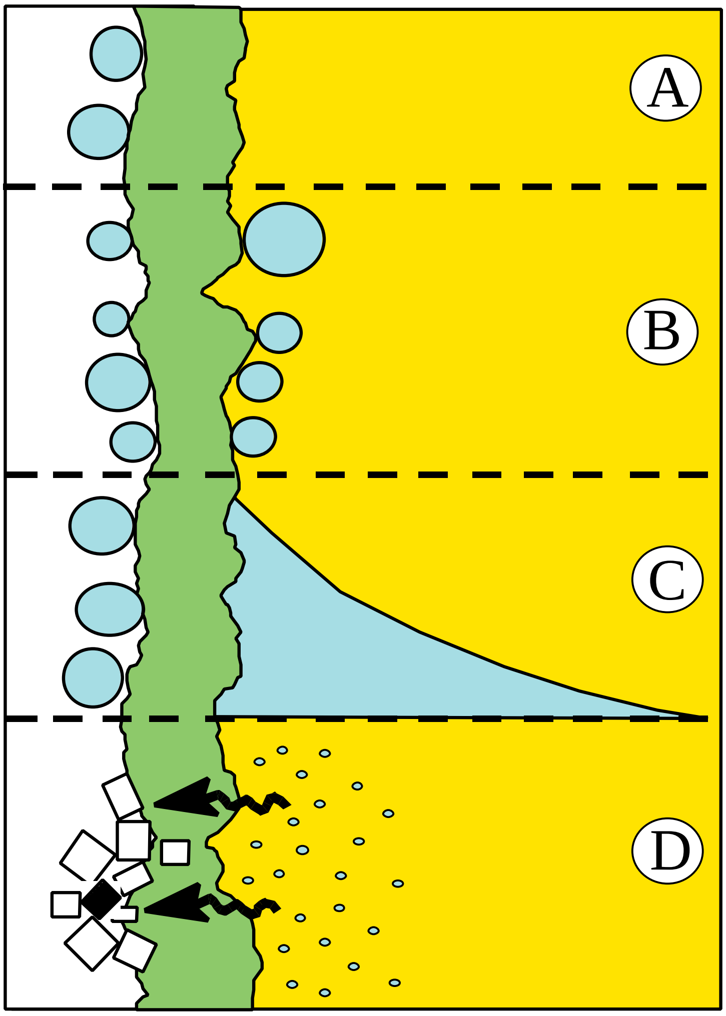


**Figure 5. Schematic section through a block of porous building material (sandstone), with a biofilm developed on the outer surface (left hand side). Key: yellow = sandstone, blue = water, green = biofilm. A) Biofilm with hydrophobic external surface. B) Biofilm with hydrophobic external and internal surfaces. C) Biofilm with hydrophobic external surface, and water ponded behind the internal surface. D) Biofilm with high humidity behind the internal surface, and salt crystals forming along the outer surface.**

However, if water is able to penetrate the structure from another direction (e.g. ingress from above, capillary rise, or possibly through condensation from within the building), it is equally possible that this water could be retained behind the biofilm (Fig. 5b). This is likely if the adhesive layer of the biofilm is hydrophobic. If this scenario occurred, the build up of water trapped behind the biofilm may accelerate the damage caused by mechanisms such as chemical leaching and frost shattering.

In extreme cases ponding of water could occur within the building stone and once again increase chemical leaching, clay swelling, salt shattering and freeze-thaw shattering (Fig. 5c).

On a more positive note, it is also possible that a hydrophilic biofilm could act as a wick and transport dissolved salts to the surface of the biofilm where they can crystallise harmlessly (Fig. 5d), thus preventing salt shattering and eventual spalling. This has already been noted by Grondona (1997).

Hydrophilic biofilms (for example comprising predominantly of algae), may act as water wicks, retaining water within the biofilm and perhaps transporting it into the porous matrix. The impact of such biofilms, at least in terms of water penetration into porous building materials such as sandstone is however likely to be limited. Most sandstones are already highly hydrophilic, being dominated by quartz, feldspar and clays such as illite, all of which are hydrophilic (Barclay & Worden 2000). Nevertheless, in cases where sandstone porosity is lined by more hydrophobic mineral cements such as calcite and dolomite, algal biofilm may have a more important role in the penetration of water into the substrate. The same will also be true for porous carbonates (such as oolitic limestone), where an algal biofilm could change pore walls from hydrophobic to hydrophilic. This would allow water to penetrate into building materials via the hydrophilic biofilm, thus bypassing pores that otherwise would have prevented or slowed down water movement through the pore space due to their hydrophobic nature. However, as algae are photosynthetic, such effects would be limited to the outer skin of any porous building material, and would therefore only effect the outer few millimetres of the building stone.

## 5 RECOMMENDATIONS

In order to assess the damaging or protective effects of the varying wettabilities of biofilms, each biofilm on a building should be examined with an open mind. A first approach to the interpretation of the biofilms wettability could be an *in situ* assessment achieved by applying small droplets of water to the biofilm. If this was not practical, a representative consortium could be grown on blocks of porous building material, under laboratory conditions, and observations made of single water droplets on the biofilm surface (Fig. 6). A second stage could involve a more detailed ESEM study of the natural biofilm, or the laboratory sample. Such an examination would determine the three dimensional architecture of the biofilm, elucidate the distribution of the various biofilm components, and define the wettability of each component present. The combined information could then be taken and used to model likely water migration pathways, and predict water distribution within the porous building material. The latter could then be used to predict likely mechanisms (involving water) that may affect the durability of that particular building material.

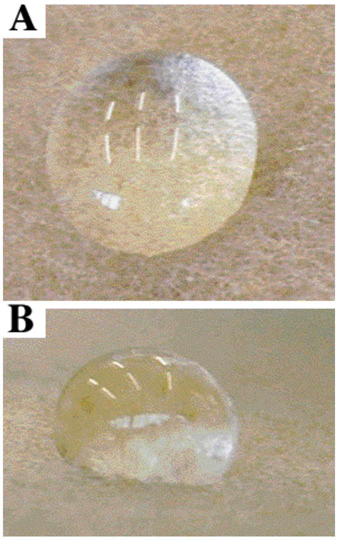


**Figure 6. A), B) Plan and side views of water droplets on a block of sandstone with a bacterial-fungal biofilm on the surface. Droplets have a high contact angle, being spherical in form, demonstrating the hydrophobic nature of the biofilm at the macroscopic scale. Droplets are approximately 2-3 mm in diameter.**

## 6 CONCLUSIONS

Biofilms of various types will readily colonise building materials. Analysis under ESEM has shown that the wettability of the underlying substrate can be dramatically altered by the presence of biofilm. The change can be towards either an hydrophilic or hydrophobic character. The implications for building materials can be either positive or negative. In some instances they can be protected from common weathering effects such as salt shattering and frost shattering, but in others the problems could be magnified. It is vitally important to asses the nature of biofilms present on buildings and monuments, just as important as assessing the properties of the building material. One must be sure if the biofilm is “friend or foe”.

## 7 ACKNOWLEDGEMENTS

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# Service Life Prediction Of Reinforced Concrete Structures Exposed To Aggressive Environments

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**Summary: In this paper, the service life of structures exposed to aggressive environments is measured by the probability of cracking and spalling of concrete cover. The time to corrosion cracking/spalling is experimentally investigated from accelerated corrosion testing of RC slabs with the emphasis on trying to quantify the relationship between concrete quality (w/c ratio; or strength), concrete cover, crack propagation and time. The probability of cracking and spalling of concrete cover is calculated by using a structural deterioration life-cycle reliability model. The reliability model includes the random spatial variability of concrete compressive strength, concrete cover and the surface chloride concentration. The reliability model also includes a stochastic deterioration model that considers the random variability of chloride diffusion, threshold chloride concentration and corrosion rates. Therefore, the reliability model can be used to predict the proportion of a concrete surface likely to spall for any reference period. This is a useful criterion for predicting the service life of RC structures.**

**Keywords: Cracking/Spalling, Structural Reliability, Corrosion, Concrete, Service Life**

## 1 INTRODUCTION

Corrosion of reinforced concrete (RC) buildings, bridges and other structures is initiated mainly by chloride contamination, often in conjunction with inadequate cover and poor quality concrete. Chloride contamination can occur from the application of de-icing salts or sea-spray. Corrosion products (rust) are expansive, leading to the formation of tensile stresses in concrete and therefore subsequent longitudinal cracking and spalling of concrete cover. Corrosion-induced longitudinal cracking and associated spalling of the concrete cover are particularly common problems in concrete structures. Cracking and spalling may result in the quicker diffusion of aggressive ions, water and oxygen thus resulting in further reduction in bar diameter, further reduction of flexure and shear capacity and therefore shortening the service life of RC structures. As such, the incidence of spalling of concrete cover (probability of spalling) is the focus of this paper.

The observation of severe cracking or spalling of concrete cover indicates the need for repair or more frequent maintenance. This is classified as a serviceability limit state. Repair or rehabilitation costs can be extensive, and such expenditure may represent the end of a “maintenance free” (cost-free) service life (Stewart & Rosowsky 1998; Stewart 1999; Thoft-Christensen 2001). However, other definitions of serviceability failure for RC structures with corroding reinforcement have been used as well, including the initiation of corrosion (e.g. Troive & Sundquist 1998), the occurrence of first cracking (e.g. Weyers 1998), or when the percentage of reinforcement subject to corrosion exceeds a certain threshold (e.g. Amey et al. 1998).

A new limit state that would specify the tolerable extent of damage (e.g. in terms of percentage of concrete cover subject to spalling) may be necessary. Such information may be used, for example, in a life-cycle cost analysis to optimise repair or maintenance strategies (Stewart 2001). This will require the development of random field modelling that takes into account the spatial variability of corrosion initiation, propagation and cracking (e.g. Vu & Stewart 2001; Sterritt et al. 2001; Faber & Rostam 2001). This is the approach adopted in this paper where the service life of a structure is measured by the probability of corrosion-induced cracking and spalling of the concrete cover considering the spatially variability of corrosion initiation, propagation and cracking. The probability of cracking and spalling is estimated herein using a structural deterioration life-cycle reliability model for typical RC surfaces exposed to aggressive environments.

In this paper, crack initiation and propagation measurements obtained from accelerated corrosion tests of RC slabs are presented. The experimental design considered variability of concrete cover, concrete strength and water-cement ratio. An empirical stochastic model for crack initiation and propagation is then developed to predict the time to severe corrosion-induced cracking and spalling. The reliability model includes this new predictive model for time to severe cracking and spalling, as well as the variability of chloride diffusion, threshold chloride concentration, corrosion rates, reinforcement placement and environmental conditions. Concrete cover, water-cement ratio and surface chloride concentration are spatially variable which enables the proportion of a concrete surface likely to spall to be calculated for any reference time period.

## 2 CORROSION INITIATION AND PROPAGATION

Corrosion is initiated when the chloride concentration at the bar surface exceeds a threshold level ($C_r$). Chloride concentration is estimated by Fick's second law of diffusion since most of the available parameters are obtained from fitting Fick' s second law to experimental or field data. The chloride diffusion coefficient (D) is modelled as a function of concrete quality and concrete cover, according to Vu & Stewart (2000) as follows:

$$D = D_{H_2O} \times 0.15 \frac{1 + r_c (w/c)}{1 + r_c (w/c) + \frac{r_c}{r_a}(a/c)} \left[ \frac{r_c (w/c) - 0.85}{1 + r_c (w/c)} \right]^3 \tag{1}$$

where a/c is the aggregate-to-cement ratio; $\rho_c$ and $\rho_a$ are the mass densities of cement and aggregates respectively; $D_{H2O}$ is the diffusion coefficient in an infinite solution (=$1.6 \times 10^{-5}$ $cm^2$/sec for NaCl), and w/c is the water-cement ratio estimated from the concrete compressive strength using Bolomey' s formula.

Middleton & Hogg (1998) and others have reported that cover and concrete quality are observed to affect corrosion rates. When relative humidity is in the region of 70-85% the oxygen availability at the cathode and the electrical resistivity of concrete are factors affecting corrosion rates (e.g. Tuutti 1982; Yalsyn & Ergun 1996; Yokozeki et al. 1997). For many structures in Europe, U.S., Australia and elsewhere the average relative humidity is over 70%. Unfortunately, there are few quantitative models for predicting corrosion rates for these environmental conditions, except that corrosion rates ($i_{corr}$) for low, medium and high corrosion intensities have been estimated to be 0.1$\mu$A/$cm^2$, 1$\mu$A/$cm^2$ and 10$\mu$A/$cm^2$ respectively (Dhir et al. 1994). To isolate the effect of concrete quality (w/c ratio) and concrete cover it is assumed herein that the corrosion rate is limited by the availability of oxygen at the steel surface (e.g. Arnon et al. 1997). As such, the oxygen availability depends on concrete quality (w/c ratio), cover and environmental conditions. Therefore, corrosion rate is modelled as a function of concrete quality, cover and time since corrosion initiation (Vu & Stewart 2000); namely,

$$i_{corr}(1) = \frac{37.8\,(1 - w/c)^{-1.64}}{cover} \quad (\mu A/cm^2) \tag{2}$$

where $i_{corr}(1)$ is the corrosion rate at the start of corrosion propagation and cover is given in mm and

$$i_{corr}(t_p) = i_{corr}(1) \times 0.85 \times t_p^{-0.29} \qquad t_p \geq 1\ year \tag{3}$$

where $t_P$ is time since corrosion initiation and $i_{corr}(1)$ is given by Eqn. 2.

Statistical parameters for corrosion parameters are given in Table 1.

**Table 1. Statistical Parameters for Corrosion**

| *Parameter* | *Mean* | *Coefficient of Variation* | *Distribution* |
|---|---|---|---|
| Model Error (D) | 1.0 | 0.2 | Normal |
| $C_0$-de-icing salts | 3.5 kg/$m^2$ | 0.5 | Normal |
| $C_0$-coastal zone (within 100m of coast-line) | 2.95 kg/$m^2$ | 0.5 | Normal |
| $C_r$ | 0.9 kg/$m^3$ | 0.19 | Uniform [0.6–1.2] |
| Model Error ($i_{corr}$) | 1.0 | 0.2 | Normal |

## 3 CORROSION-INDUCED CRACKING AND SPALLING OF CONCRETE COVER

The incidence of cracking and spalling is more frequent than "collapse" and other ultimate strength limit states. The occurrence of longitudinal cracking and spalling of the concrete cover is referred to herein as a serviceability failure, and if not repaired may often be precursors to more critical and dangerous strength limit state problems. Of more relevance, however, is that the observation of cracking and spalling will normally require a re-assessment of structural integrity, more frequent inspections and the possible need for repairs – these will require the outlay of additional financial resources. In this study, the time to corrosion-induced cracking and spalling is studied by accelerated corrosion testing of RC slabs.

### 3.1 Accelerated Corrosion Testing

3.1.1 Materials and Specimens

In order to achieve the purpose of predicting the time to longitudinal cracking and spalling, experimental studies were conducted at The University of Newcastle, where typical RC specimens (slabs) were subject to accelerated levels of corrosion. The accelerated corrosion experimental programme is designed to simulate the corrosion of a section of a typical bridge deck structure. The tests consisted of two series. The first series of tests comprised of four specimens (two specimens had 25mm cover, the other two had 50mm cover). All specimens had the same water-cement (w/c) ratio (w/c=0.5), but differences in mix designs resulted in different concrete strengths. The second series of tests isolated the effect of w/c ratio and cover. These specimens had w/c=0.45 and w/c=0.58 and 25mm and 50mm covers. All specimens were 700mm x 1000mm rectangular slab with thickness of 250mm. The top mat of the slab contained four steel reinforcing bars, which were covered with electroplating tape to give exposed (bare steel) lengths of 1000mm.

Ordinary Portland cement was used in the mix and 3% of $CaCl_2$ by weight of cement were added to the concrete mix in order to induce corrosion along the length of exposed bars. The specimens were moist-cured for 28 days before testing. Standard test cylinders were tested at 28 days to determine concrete compressive and tensile strengths.

3.1.2 Testing Methodology

The active accelerated corrosion process was achieved by applying an electrical current to the bars. The accelerated corrosion equipment is shown in Fig. 1. The soffit of the specimen was immersed in a 5% NaCl solution. A current was then supplied to the bar (the anode) by a power supply via a current regulator (LM317LZ Voltage regulator IC one per bar), and the cathode was a stainless steel plate submerged in the NaCl solution. The current regulator kept the current constant over time, in this case equivalent to a corrosion rate of $100\mu A/cm^2$. This is equal to the highest corrosion rate recorded in concrete structures (most corroding structures experience 1-2 $\mu A/cm^2$). This high current allows a short period of testing, but kept the corrosion rate within the highest limit of the natural values found in real structures.

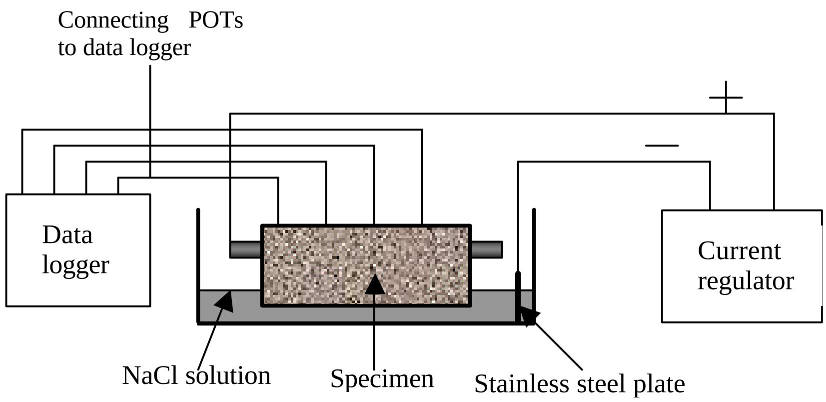


**Figure 1. Experimental Set up of Accelerated Corrosion Test**

The appearances of first visible cracking were detected by crack detector gages, which were installed on the concrete surface above the bars and also were verified by frequent visual observation using a magnification glass with an accuracy of 0.05mm. Concrete surface deformation before first visible cracking was also measured by strain gages.

Crack development after the first appearance of visible cracks was recorded using 10mm linear potentiometers displacement transducers (POTs - Sakae 8FLP10A 1000ohm conductive film potentiometer), which were glued on both sides of the crack. There were in total 12 POTs on each specimen to measure crack development. Hence, the crack width measurements presented herein is the mean value of data read from 12 POTs. The data collected for crack propagation will provide the timing of first cracking and the subsequent time-dependent increase in crack width. After testing, the weight loss of bars due to corrosion were studied by cleaning, drying and weighing the reinforcement bars according to the gravimetric weight loss method as specified by Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens (ASTM G1 – 90). The weight loss corresponded closely to that expected from $i_{corr}$ measurements.

### 3.2 Results from Accelerated Corrosion Tests

3.2.1 Crack Initiation ($t_{1st}$)

The term -“first cracking”- used herein is the first visible crack, which was observed through a magnifying glass (hairline crack of width less than 0.05mm) and this period of time ($t_{1st}$) can be referred to as time to crack initiation. Test results indicated that $t_{1st}$ depends on concrete tensile strength ($f_t$), w/c ratio and cover (C). The test results suggest that the model developed by Liu

& Weyers (1996) reasonably predicts the time to first cracking although the predicted times are under-estimated (approximately 11%) when compared to results obtained from accelerated corrosion testing.

In the Liu & Weyers (1996) model the time to crack initiation is the time when stresses resulting from the expansion of corrosion products exceed the tensile strength of concrete. The critical amount of corrosion products needed to cause first cracking consists of two parts: (i) the amount of corrosion products required to fill the total porous zone around the steel/concrete interface; and (ii) the amount of corrosion products then needed to generate the critical tensile stresses. The time to cracking is influenced by corrosion rate, cover, bar spacing, concrete quality, and material properties. Therefore, the model proposed by Liu & Weyers (1996) is assumed herein as suitable for estimating the time to first cracking.

Note that these accelerated corrosion tests simulate localised (pitting) corrosion to be expected with chloride-induced corrosion. This is an important observation since the Liu & Weyers (1996) model appears suitable for this type of localised corrosion (i.e., widespread along the length of reinforcing bars) even though this and other corrosion-induced cracking models were developed on the assumption of general corrosion

#### 3.2.2 Crack Propagation ($t_{ser}$)

Cracks first observed at the concrete surface through the magnifying glass were very small (width of less than 0.05mm) with lengths varying from 30mm to 300mm, with the most common length being 120mm to 150mm. Cracks width and length then increased in an inhomogeneous manner until they extended and joined together to create continuous corrosion-induced longitudinal cracking when the crack width is about 0.4mm to 0.5mm.

The results show that crack propagation increases linearly with time for crack widths less than 0.5mm. After this period rate of crack-growth seems to reduce and change to a non-linear trend. The reduced rate of crack propagation when crack width exceeds a certain value (0.5mm) may be attributed to the fact that the corrosion products can now easily diffuse through the crack toward the concrete surface, and so no longer contributes solely to the build up of expansive tensile pressures around the bar. In fact, at this time rust (corrosion product) with dark red-brown colour was clearly observed on several locations on the specimens' surfaces.

There are different views about the limit crack width. For example, Andrade et al. (1993) pointed out that a limit crack width between 0.3 to 0.4mm is appropriate for a serviceability limit state. On the other hand, Sakai et al. (1999) stated that a limit crack width of 0.8mm is recommended for serviceability (aesthetics) requirements. Recall that experimental data showed that cracks were joined together to create longitudinal cracking at crack widths of about 0.4 to 0.5mm. At this stage, spalling would likely occur in a real slab with transverse reinforcement and where there are loads applied to the structure. This is consistent with the general observation that the service life of a structure is reduced considerably only if cracks with widths exceeding 0.3-0.5mm are not repaired (Andrade et al. 1993). Hence, in the present paper, the limit crack width is taken as 0.5mm and so time to severe cracking ($t_{ser}$) is referred to herein as the time for the crack to propagate from 0.05mm to 0.5mm.

### 3.3 Influence of Concrete Cover (C)

All of the experimental slabs had been designed for the purpose of studying the effect of concrete cover, with the cover being either 25mm or 50mm. As expected, it is observed that concrete cover influenced crack propagation for experimental slabs at 25mm and 50mm at different w/c ratios (i.e. 0.45; 0.5; 0.58) (see Fig. 2). It is noted that the cracking patterns at the cross sections were quite similar for both 25mm and 50mm cover. It is observed that the crack propagation time ($t_{ser}$) at 50mm cover is about 1.15; 1.2; 1.4 times that observed for 25mm cover slabs at 0.45; 0.5; 0.58 w/c ratios, respectively. However, the effect of concrete cover was not significant when crack widths were less than 0.15mm to 0.25mm.

### 3.4 Influence of Concrete Water-Cement Ratio (w/c)

The influence of w/c ratio was studied by keeping slabs at the same cover but having different w/c ratios. It was observed that increasing w/c ratio resulted in increased crack propagation rates by up to 30% and 40% for 25mm and 50mm cover respectively (see Fig. 2). However, as was observed for concrete cover, the w/c ratio appears to mostly influence crack propagation when the crack width exceeds 0.15mm to 0.3mm.

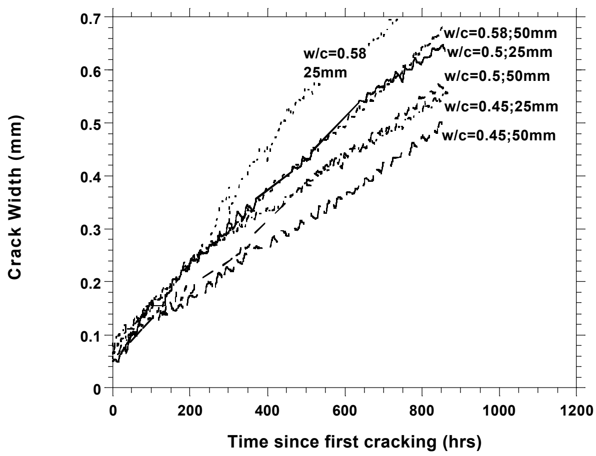


**Figure 2. Crack Propagation Since First Cracking**

### 3.5 Influence of Concrete Strength ($f'_c$ or $f_t$)

It appears that, for these test conditions, concrete strength did not influence crack propagation for crack widths less than 0.5mm. It is reasonable to expect that tensile strength may only influence the tensile stresses needed to cause first cracking (Liu & Weyers 1996; Williamson & Clark 2000). However, it was observed that crack propagation depends on concrete strength only when the crack width is larger than 0.5mm. Thus, concrete strength may not be an important consideration for this study since in reality repairs would normally be undertaken for cracks of about 0.3mm to 0.5mm.

### 3.6 Combined Effects of Concrete w/c Ratio and Cover

The effect of "concrete quality" is studied by including both the effects of w/c ratio and cover. For the present paper, "concrete quality" is categorised as: (i) best quality is 50mm cover and w/c=0.45; (ii) fair quality could be any of the following: 25mm;w/c=0.5 or 50mm;w/c=0.58 or 50mm;w/c=0.5 or 25mm;w/c=0.45; (iii) worse quality is 25mm cover and w/c=0.58. This represents a general quality specification in real situations, although the specific concrete qualities in real structures will most likely depend on other factors. If combining the effects of w/c ratio and cover together, it can be noted that a new parameter "concrete quality", defined as the ratio between w/c ratio and cover (or wc/C) is an important factor controlling the time to cracking and spalling. As seen in Fig. 3, a longer time for crack propagation occurs for best quality concrete, while a lower quality concrete results in increased crack propagation rates.

It is possible to suggest from Fig. 3 that there is a non-linear relationship between $t_{ser}$ and wc/C derived empirically as

$$t_{ser} = \mathrm{A} \times 10^{-3} \times \left(\mathrm{wc/C}\right)^{-\mathrm{B}} \tag{4}$$

where wc/C is the ratio between w/c ratio and cover (C); $t_{ser}$ is the time since crack initiation (years); and A and B are obtained from regression analysis of experimental data depending on crack width (for 0.5 mm crack width, A=6.5 and B=0.57). It is noted that, Eqn. 4 only applies for crack widths greater than 0.3mm.

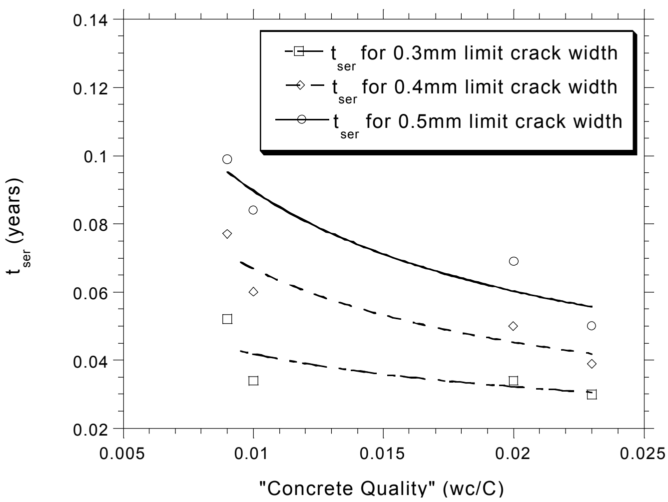


**Figure 3. Effect of Concrete w/c Ratio and Cover on Crack Propagation**

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### 3.7 Predictive Model of Time-Dependent Crack Propagation

The various stages of time-dependent crack propagation are represented in Fig. 4. The time to cracking and spalling referred to herein is the time when concrete cover cracking reaches a limit crack width of 0.5mm. Therefore, time-dependent crack growth can be divided into two stages:

(i) Crack initiation - influenced by many factors such as concrete cover, concrete properties (w/c ratio and strength), bar diameter;

(ii) Crack propagation - mainly dependent on concrete cover, w/c ratio, and concrete strength.

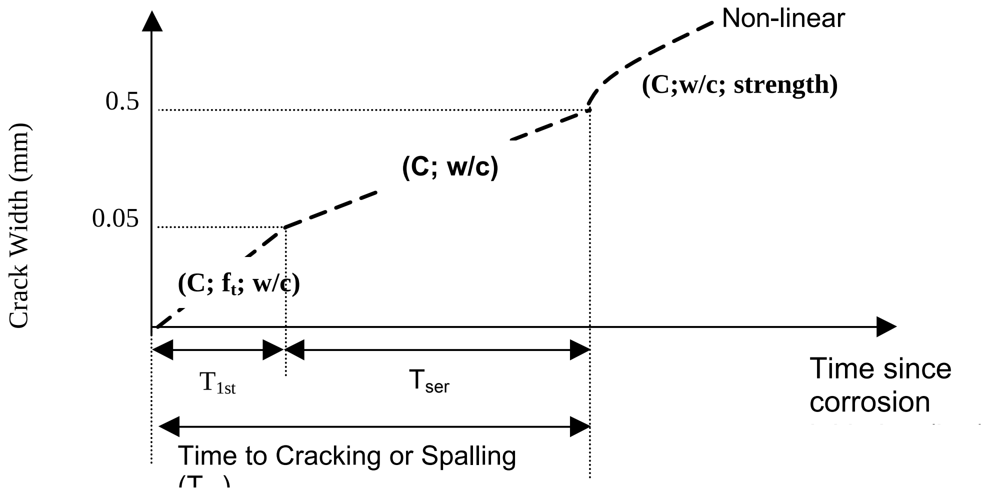


**Figure 4. Modelling Time Dependent Crack Growth**

In the context of this paper, only crack widths less than 0.5mm are of direct interested as it can be assumed that under these condition cracks propagate linearly with time and only depends on concrete cover and w/c ratio. The models proposed herein are preliminary only. Work is continuing to develop physically-based predictive models (as opposed to the empirical or "best-fit" model shown herein) suitable for predicting crack propagation up to any crack size (not limited to 0.5mm). The definition of what constitutes a limit crack width in the context of service life prediction is also required.

## 4 PROBABILITY OF LONGITUDINAL CRACKING AND SPALLING

For any time-invariant corrosion rate the time to severe cracking ($t_{ser}$) is modified from Eqn. 4 such that

$$t_{ser} = \left[A \times 10^{-3} \times (wc / C)^{-B}\right] \times \frac{100}{\mathrm{i}_{\mathrm{corr}}(1)} \tag{5}$$

where $i_{corr}(1)$ is time-invariant corrosion rate estimated from Eqn. 2; and A=6.5 and B=0.57 for a limit crack width of 0.5mm.

For time-variant corrosion rates, the time to cracking and spalling is $T_{sp} = T_{1st} + T_{ser}$ where $T_{1st}$ and $T_{ser}$ are time to crack initiation and time to severe cracking respectively for time-variant corrosion rates such as that given in Eqn. 3. Note that the upper case "T" denotes crack initiation and propagation times obtained for time-variant corrosion rates, while the lower case "t' refers to times obtained using time-invariant corrosion rates. Hence, $T_{sp}$ will be greater than $t_{sp}$.

If it is assumed that (i) the amount of rust produced until severe cracking is the same for time-variant and time-invariant corrosion rates; (ii) the time-variant reduction in corrosion rate is given by Eqn. 3; and (iii) corrosion rate for the first year is not time-variant, hence for the first year $T_{1st}=t_{1st}$, then $T_{sp}$ can be estimated as

$$T_{sp} = \mathrm{T}_{1st} + \mathrm{T}_{ser} = \left[0.84\left(\mathrm{t}_{ser} + t_{1st} + 0.2\right)\right]^{1.4} \quad (\mathrm{t}_{1st} + \mathrm{t}_{ser} > 1 \text{ year}) \tag{6}$$

In the present study it is proposed that $t_{1st}$ be obtained from the Liu & Weyers (1996) model and $t_{ser}$ is estimated from Eqn. 5. In both cases, $t_{1st}$ and $t_{ser}$ are calculated based on time-invariant corrosion rates.

Then, the probability that longitudinal cracking and spalling ($F_S$) will occur at least once during the time interval T is defined herein as

$$F_S(T) = \Pr\left(t > T_i + T_{sp}\right) \tag{7}$$

where $T_i$ is the time to corrosion initiation. This represents a first passage probability.

## 5 MODELLING OF RANDOM SPATIAL VARIABILITY

In practical applications of structural reliability analysis for corrosion in RC structures, there are many parameters that vary in space and such random spatial variability can be modelled as a random field. The probabilistic analysis of structures considering random spatial variability involves the discretisation of the corresponding random fields into sets of spatially correlated random variables. Hence, the structure is divided into n elements, and a random variable is used to represent the random field over each element. The statistical correlations between the random variables for different elements are based on the correlation characteristics of the corresponding random field (Vanmarcke 1983). In this paper, the spatial averaging method suggested by Vanmarcke (1983) is used to model the random spatial variability of concrete property and the surface chloride concentration. The analysis considering random spatial variability is described below, for more details of the spatial variability analysis see Vu & Stewart (2001).

### 5.1 The Variance Function

The spatial averaging method represents the random field X(t) over an element i is

$$\mathrm{X}_{\mathrm{i}}(t) = \frac{1}{T}\int_{t-T/2}^{t+T/2} X(t)dt \tag{8}$$

where T is the averaging interval and X is the parameter of interest.

The mean value is not affected by the averaging operation while the variance of $X_i(t)$ will be changed and may be expressed as

$$Var\left[X_i\right] \equiv s_{\mathrm{i}}^2 = g(\mathrm{T})s^2 \tag{9}$$

where $\gamma(T)$ is the variance function of $X_i(t)$ and $\sigma_i$ is the standard deviation of $X_i(t)$

### 5.2 Correlation Between Local Averages

The covariance matrix, which describes the correlation between two elements i and j is estimated as

$$COV\left\lfloor X_i, X_j \right\rfloor = \left(s_{\mathrm{i}} s_j\right) \times r_{\mathrm{X}_i, X_j} \tag{10}$$

where $\rho_{Xi,Xj}$ is the coefficient of correlation between the local averages $X_i$ and $X_j$ and is expressed linearly as\

$$r_{X_i,X_j} = \frac{1}{2}\left[\mathrm{T}_0^2 g(T_0) - T_1^2 g(T_1) + T_2^2 g(T_2) - T_3^2 g(T_3)\right] \tag{11}$$

where $T_0$ is the distance from the end of the first interval to the beginning of the second interval; $T_1$ is the distance from the beginning of the first interval to the beginning of the second interval; $T_2$ is the distance from the beginning of the first interval to the end of the second interval; $T_3$ is the distance from the end of the first interval to the end of the second interval.

### 5.3 Random Field Analysis and Simulation

In this study, a one-dimensional random field is investigated for concrete compressive strength, concrete cover and surface chloride concentration. The variance function $\gamma(T)$, which measures the reduction of the point variance $\sigma^2$ under local averaging, is related to the correlation function $\rho(\tau)$. In this paper, the correlation function is a triangular type correlation function that decreases linearly from 1 to 0 as $|\tau|$ goes from 0 to $\theta$, where $\theta$ is the scale of fluctuation. It is assumed herein that $\theta$ = 0.5m for concrete compressive strength, cover and the chloride surface concentration (Vu & Stewart 2001).

To model the spatial variability of concrete compressive strength, cover and the chloride surface concentration, the structure is discretised into n rectangular elements - whose edges are parallel to the coordinate axes. Once the stochastic random field is defined, Monte Carlo simulation methods can be used to randomly generate parameter values for each of the n discretised elements.

## 6 ILLUSTRATIVE EXAMPLE – STOCHASTIC MODELLING OF BRIDGE DECK DETERIORATION

The following example will help illustrate the effect of “concrete quality” on corrosion-induced cracking and spalling of concrete cover, which can then be used to assess the service life of RC structures. For this study, chloride contamination will occur from exposure to sea-spray (for structures located within 100m of the coast) or the application of de-icing salts.

The bridge deck considered in this study has a span length of 11m and a width of 14.2m. The bridge design required a 550mm thick slab with top and bottom steel. The diameter of longitudinal reinforcing steel (top) is 15mm and bar spacing is 165mm. Concrete cover varies from 25mm to 75mm. Statistical parameters for dimensions and material properties appropriate for this RC bridge deck are given in Table 2.

The slab deck has been discretised one-dimensionally into 8 elements of equal length. Since the probabilities of failure are expected to be high Monte-Carlo simulation was used for the analysis. The simulation analysis includes:

- stochastic deterioration model (Table 1);
- random field modelling for concrete cover, w/c ratio and surface chloride concentrations;
- new predictive model for time to cracking and spalling – Eqn. 6; and
- probability that cracking and spalling will occur at least once for any reference time.

This means that dependent variables such as concrete tensile strength, corrosion rate, and crack initiation and propagation are also spatially variable.

**Table 2. Statistical Parameters for Material Properties and Dimensions**

| *Parameters* | *Mean* | *Coefficient of Variation* | *Distribution* |
|---|---|---|---|
| Top cover | $D_{tnom}$+19.8mm | $\sigma$ = 16.5mm | Normal |
| $f'_{cyl}$ | $f'_c$+7.5MPa | $\sigma$ = 6mm | Normal |
| $f'_{ct}(t)$ | $0.53(f'_c(t))^{1/2}$ | 0.13 | Normal |
| $E_c(t)$ | $4600(f'_c(t))^{1/2}$ | 0.12 | Normal |
| $k_w$ ($f'_c(28)=k_w f'_{cyl}$) | 0.87 | 0.06 | Normal |

The final results are presented in terms of the proportion of a concrete surface likely to spall for any reference period. Figure 5 shows the effect of concrete cover and w/c ratio on the percentage of deck spalled for top cover, for sea-spray over a 120 year service life. The percentages of spalling for exposure to de-icing salts are similar to that shown in Fig. 5 since for this example time to initiation is relatively short and differences between the surface chloride concentrations are small (see Table 1).

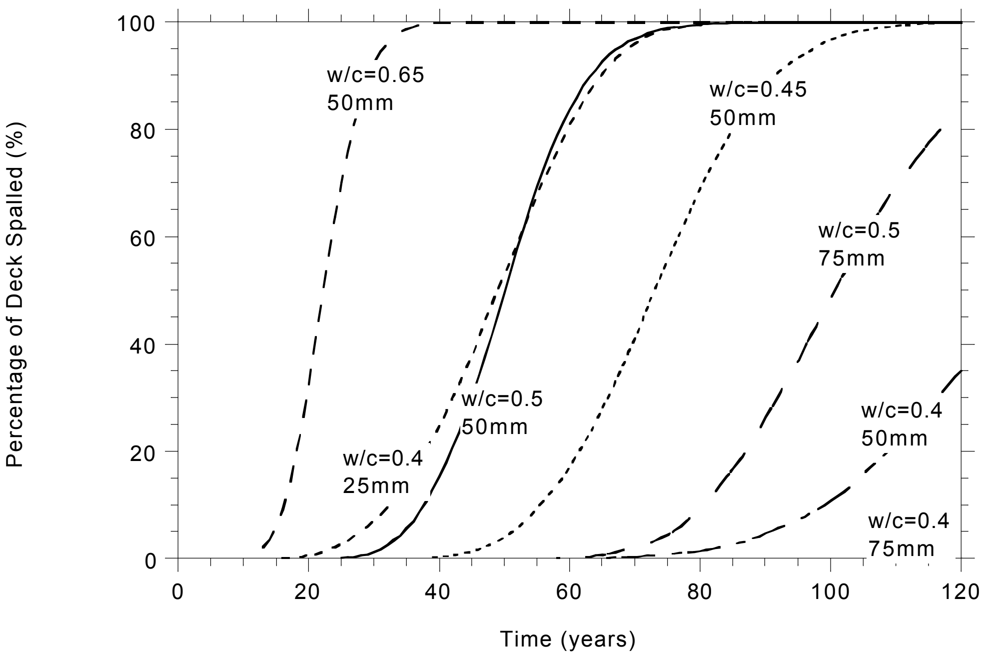


**Figure 5. Effects of Concrete Cover and w/c Ratio on Concrete Cracking and Spalling for Sea-spray**

As shown in Fig. 5, for 50mm cover and w/c=0.65 the percentage of deck spalled will be 100% after only 40 years. While for the same cover, reducing w/c to 0.5 will result in 100% spalling after 80 years of service and no spalling anywhere on the deck after 65 years of service for w/c=0.4. This helps quantify the very important role of concrete cover and w/c ratio on the service life performance of RC structures exposed to aggressive environments. Clearly, the representation of results in terms of the proportion of a concrete surface likely to spall for any reference period is information that can be used in a life-cycle cost analysis to optimise durability design requirements for new structures or optimise repair/maintenance strategies for existing concrete structures.

Alternatively, Fig. 6 shows the effect of modelling corrosion rate as a time-invariant variable – i.e. Eqn. 2. Not surprisingly, modelling corrosion rate as a time-invariant variable results in a considerable increase in the likelihhod of cracking and spalling for any reference period. This indicates the sensitivity of results to the chosen stochastic model of corrosion rate. Clearly, the results will also be sensitive to the stochastic models used for crack initiation and propagation.

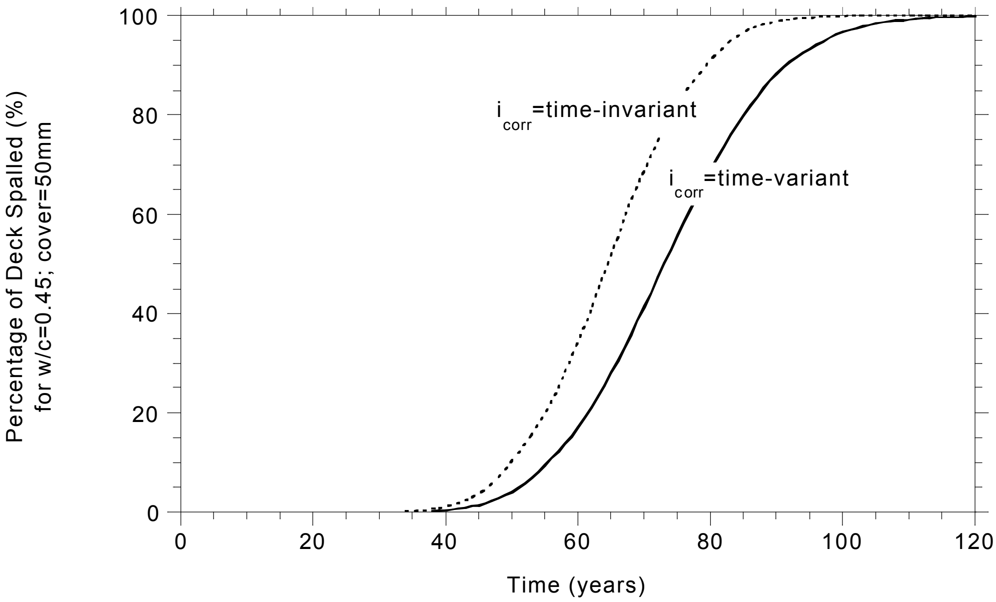


**Figure 6. Effect of Modelling Corrosion Rate as a Time-Invariant Variable, for 50mm cover and w/c = 0.45.**

## 7 CONCLUSION

The paper has developed stochastic models for crack initiation and propagation obtained from accelerated corrosion testing of RC slabs. The present analysis then included the modelling of the deterioration process and subsequent likelihood of cracking or spalling for RC structures considering concrete cover, w/c ratio and surface chloride concentration as spatially variable. Other dimensional, material and deterioration parameters were treated as random variables. This allows the proportion of a concrete surface likely to crack and spall for any reference period to be estimated. The results indicate that "concrete quality" specifications (cover, w/c ratio) are important in controlling the service life of RC structures exposed to aggressive environments. However, further research is progressing to gain a better understanding of the effects of concrete properties, concrete cover and the surface chloride concentration have on cracking and spalling of concrete cover, as well as the development of improved stochastic models for corrosion initiation, corrosion propagation and crack initiation and propagation.

## 8 ACKNOWLEDGEMENTS

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# Design Decisions For Durable Roofing

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**Summary: Roof systems have been designed and built for thousands of years. Although we have seen many changes in the types of materials used in roof systems over the last 30 years, not much has changed in the basic design of these systems. So why is it that we continue to design and build low-slope roofs with little or no slope, inadequate drainage, and service lives as short as 10 years? Has initial cost become the only controlling factor in roof construction? There is plenty of published data regarding the expected service life of a multitude of different roof types. The data suggests that there are roof systems with an average service life as short as 10 to 15 years. With over 20 years of experience in analyzing the performance of roof systems, the author presents some key design factors that are often overlooked or rejected when choosing a new or replacement roof system. If the designer were to consider these factors and choose more carefully, then roof system service life would undoubtedly increase. This paper outlines a common-sense approach to design that integrates the concepts of slope, drainage, attachment, material durability, constructability, and maintenance. The roof designer should consider each of the design factors outlined in this paper in order to increase the overall durability of the roof system, and thus increase service life. The increase in durability comes, in part, from the elimination of poor roof design practices. Additionally, consistent adherence to sound drainage principles, careful material selection, matching the material selection to the building and deck substrate, and attention to construction and maintenance issues can increase expected service life. If a roof system is well constructed by a reputable roofing contractor and designed with an optimization of these six key design factors in mind, there is no reason not to expect a roof service life of 20 to 30 years. That should be the goal in building design. After all, our buildings are rarely designed to stand for only 10 to 15 years. Why have we come to expect such short service life from the roof system?**

**Keywords: Drainage, attachment, durability, constructability, maintenance**

## 1 INTRODUCTION

Roof systems are a basic part of man's shelter and roofs have been designed and built for thousands of years. Primitive shelters were built with the simple approach of creating slope in the roof to allow water to shed. Construction technology has advanced significantly over the last 30 years and we have seen many changes in the types of materials used in roof systems. Roof slope has also diminished in order to economize on the structural shapes that help build significant slopes into our structures.

Initial cost appears to have become the controlling factor in roof construction. Roof service life has been shortened in the process. New materials utilized in the 70's and 80's have been shown to have significantly shorter service lives than more traditional built-up roof products (Cash 1997). However, weather resistant properties of materials have also improved, and longer service life can be expected as we learn from our mistakes and material and application technology improves (Hoff 1997).

In the process of introducing new roofing materials and blending old and new technologies, the basic rules of sound design have not changed. A common sense approach to low slope roof design can lead to increased service life and more successful roofing and roof replacement projects. This approach to more durable roof systems integrates the concepts of slope, drainage, attachment, material durability, constructability, and maintenance.

If a roof designer pays close attention to these six key factors, a durable, economical, long-lasting roof system can be designed for almost any structure. This may not sound like a revelation, but it is a departure from the normal design process in the United States. In the United States, the roof system selection process is often driven by cost, and is driven toward using specific types of roof products and adapting those products to the widest range of building systems possible.

Each structure has, or will have, its own set of criteria that should lead the designer to choosing a system that's right for the building. That may mean that each building will have an optimal attachment method, set of material durability characteristics, construction methods, and maintenance characteristics that need to be carefully considered in choosing a roof system. The application of one type of roof system just won't apply in all cases.

After all, that's what smart design is all about. In this paper, the author uses the experience of over 20 years of observing roof system performance, gathers much of the design wisdom that has been in the roofing industry for many years, and assembles it into an organized thought process that leads to better roof system design choices. These design choices take into account the conditions a roof system is meant to meet and overcome. By considering the six key factors outlined in this paper, a roof designer will be led to a system more compatible with the structure, able to meet the challenges of the climate, with ease of construction and maintenance in mind. Such a process, if ever systematized and adapted to a computer database, may lead to "smart design" of roof systems with the aid of computer technology.

## 2 SLOPE

Slope is a very important factor in the proper functioning of a roof system. Adequate slope can compensate for inferior materials and construction quality. Most primitive roof systems relied primarily on slope to provide protection from the elements. In early structures, roof system materials often took a secondary role in the construction of roof systems, and roofs were most often constructed with materials commonly available (Vogel *et al*, 1986). The waterproofing ability of the material was often secondary to providing protection by draining water quickly away from the roof surface.

The concept of always providing roof slope seems to have been lost in the United States in the late 20$^{th}$ century. Since there were no code requirements governing roof slope in the Unites States until the late 1990's, thousands of buildings were constructed with virtually no roof slope. Roof assemblies were required to weather the burden of ponded water, often without the protection of redundant layering of membrane material. Roof membranes tended to become thinner when the use of single-ply roofing became a popular way to reduce the cost of roof systems and reduce the reliance on asphalt-based products. This was taking place at the same time that designers were flattening roof systems to save on construction cost.

The savings in construction cost has been shown to be marginal by some authors. C.W. Griffin, in the "Manual of Built-up Roof Systems", provides a strong case for the economic savings of designing roof slope into a building during construction (Griffin 1982). Griffin considers the cost of shortened service life, the inconvenience of premature tear-off and replacement, and the loss of energy efficiency due to water invasion into the insulation to far outweigh the savings of building a flat roof deck.

Although the argument was written many years ago, not much changed until building codes were written to require roof slope in new buildings. The current BOCA code in the United States requires a minimum roof slope of ¼ inch per foot or about 2% (BOCA 1999). The National Roofing Contractors Association (NRCA) recommends positive drainage and suggests criteria for judging proper slope. The criteria is "that there be no ponding water on the roof 48 hours after a rain during conditions conducive to drying" (NRCA 2001). The NRCA also recommends that the designer consult the code regarding slope requirements. This seemingly minor change in the construction code to require a minimum slope should lead to millions of dollars in savings by extending the service life of roof systems. There have been studies that show an increase in service life in traditionally flat roof systems due to positive roof drainage through roof slope (Griffin 1982).

In roof replacement situations, advances in cutting technology and design layout of tapered insulation systems have positively contributed to providing roof slope in roof replacement situations where a flat roof deck is present. Nowadays, it is a rare situation that some type of tapered insulation cannot be designed and fit to an existing roof system. Although edge heights are often increased and this may lead to flashing complications, the addition of slope is a most positive factor and the complications can generally be overcome when providing positive slope.

If one needs to be further convinced of the wisdom of providing positive roof slope, one only needs to look at residential roofing to see the positive effects. The predominant residential roof in the United States is asphalt shingle applied to a steep-sloped roof deck (Good 2001). The average homeowner is likely to experience far less problems and have a significantly longer service life from a steep-sloped shingled roof system than the average building owner with a flat roof. Warranty periods offered for steep-sloped shingle applications are generally much longer than those offered for commercial low-slope applications. In addition, the cost of steep-sloped roof coverings are often only a fraction of the cost of an insulated flat roof assembly.

The importance of the first key factor in common-sense design is to always provide positive slope to drain. A slope of about ¼ inch per foot (2%) has been shown repeatedly to work well in low-slope roof situations. Adhering to this design basic is a key factor in designing longer lasting, more durable roofs.

## 3 DRAINAGE

Roof drainage and slope are two distinctly different things. An adequately sloped roof can have inadequate drainage, and a flat roof deck can have adequate drainage. Inadequate drainage on a well-sloped roof can lead to slower water runoff, taxing the roofs' flashing systems and potentially leading to roof leaks. A relatively flat or dead level roof system can have adequate drainage, which becomes a requirement when designing protected membrane assemblies.

Although the first key factor (slope) is critical, the roof designer cannot always achieve deck slope in roof replacement situations and may have to rely on superior materials to stand up to ponded water and consistently wet situations with a flat deck. This is a common occurrence when replacing an older, flat roof with a protected membrane assembly. Adequate drainage becomes a more critical factor as roof slope approaches dead level.

Simply put, drainage refers to the ability of the roof system to carry water away. The building code usually dictates the minimum level of drainage required for a roof system. Conventional roof design calls for at least four drains for larger roofs, and a minimum of two drains for roofs under 10,000 square feet (BOCA/Plumbing Code 1990). Griffin also recommends a maximum spacing of drains in any direction of 75 feet (Griffin 1982).

The building or plumbing code provides more detailed guidelines, including the maximum tributary area per drain, as well as guidance on the required size of the drain leader. Most codes have been adapted to local rainfall rates.

Conventional design should use code-required drainage provisions and code-required overflow provisions. Although codes may not normally require upgrades to overflow protection for roof replacements, common sense design would dictate that overflow protection should be added wherever possible.

The second key factor in common sense roof design (drainage) is a simple design element in new building construction and code requirements should be followed without fail. Adding drainage to an existing roof can be challenging, but the techniques for improving drainage are numerous and there are nearly always ways to improve the situation. The addition of new internal drains, scuppers and downspouts, gutters, sumps, and tapered edges are several ways to improve roof drainage.

## 4 ATTACHMENT

In the author’s opinion, attachment is the single most critical factor that drives the choice of a new or replacement roof system. Attachment is a critical design factor in allowing a roof system to stay in place under wind loads, and most design codes have very specific requirements for wind uplift resistance.  Designers often abuse attachment methods by force-fitting a particular attachment method to a deck that does not readily lend itself to the method.

Figure 1 shows the three most commonly used roof attachment methods. The fourth method shown, the Protected Membrane Assembly, is a hybrid of the basic attachments in that the roof membrane may or may not be fully adhered, but the insulation clearly requires the ballast to keep it in place.

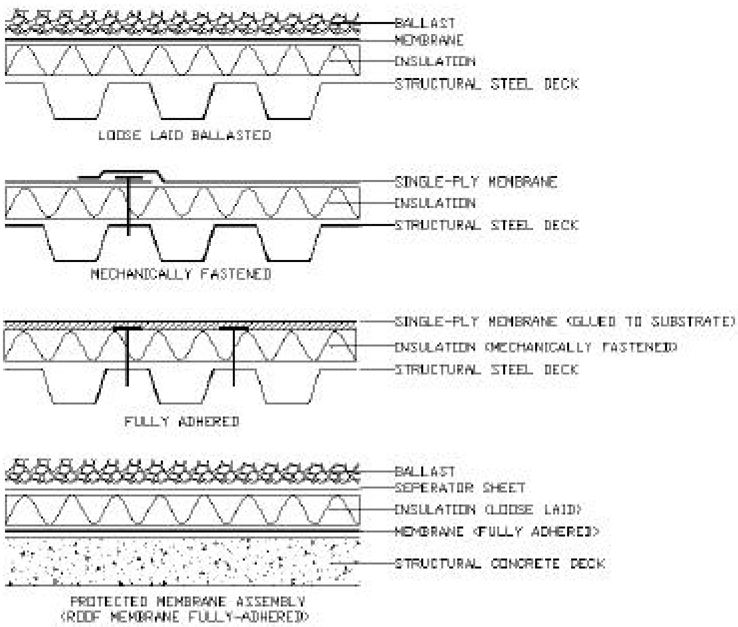


**Figure 1.  Attachment Methods**

Abuse of the attachment methods in roof design occurs when the designer does not respect the differences between nailable and non-nailable deck systems. It has been widely recognized in the roofing industry that these two major classifications of roof decks exist. Roofing manufacturer literature has generally classified roof decks as nailable or non-nailable for well over 30 years. The nailable or non-nailable designation refers to the decks ability to accept a “standard” fastener.  In the case of steel decks, self-taping screws would be the fastener of choice. In the past, hot asphalt attachment of rigid insulation to a steel deck was an accepted practice in the United States. This is no longer the case and mechanical fasteners are required to achieve appropriate wind uplift resistance ratings.

The National Roofing Contractors Association in the United States classifies roof deck systems into three categories; nailable, insulated, and concrete roof decks (NRCA 2001). Concrete roof decks are further classified into cast-in-place, precast or pre-

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stressed concrete panels, and post-tensioned concrete. The "non-nailable" designation does not appear in current NRCA design guidelines.

In the case of lightweight insulating fill and other types of lightweight deck systems, auger, splayed-shank, or some other type of hook-type mechanical fastener that grips the fill would secure the base sheet or insulation.

In the case of concrete decks, only a specialty expansion anchor or other type of expanding fastener would be effective as a mechanical attachment for insulation to concrete. However, mechanical attachment to a concrete deck system is often difficult and expensive. Most concrete decks would utilize fully adhered (requiring hot asphalt or cold adhesive attachment), or ballasted attachment methods.

Designers often attempt mechanical fastening to concrete and other non-nailable deck systems. This has led to the use of mechanical fasteners into structural concrete deck systems without regard for alternative, less expensive, and more effective attachment methods. Common problems with mechanical attachment to non-nailable decks are fastener back-out, membrane damage, and loss of uplift resistance. Ultimately, each roof type has a compatible fastening system. It is up to the designer to choose a system that has a proven track record in meeting code-required wind-uplift requirements and minimizing material incompatibility issues such as fastener back-out.

Table 1 is an approach to choosing a compatible fastening system for each of the common types of low-slope roof membrane and roof deck systems. This table is not intended to show all possible attachment methods, or even to show all commonly accepted practices in each region of the country or the world.

**Table 1. Recommended Attachment Methods for Each Major Roof Type**

| 4.1.1 Insulated Systems | | *Attachment Method by Major Deck Type* | | |
|---|---|---|---|---|
| **Roof Membrane Type** | | Steel | Concrete | LIF |
| - Built-up | Insulation | MF | FA | MF/BS – FA |
| | Membrane | FA | FA | FA |
| - Modified Bitumen | Insulation | MF | FA | MF/BS – FA |
| | Membrane | FA | FA | FA |
| - Single-ply/ Sheet | Insulation | MF or LL [1] | FA or LL [1] | MF or LL [1] |
| | Membrane | MF, FA or LLB | FA or LLB | MF, FA or LLB |
| **Membrane Direct-to-deck (Protected Membrane Assembly)** | | *Attachment Method by Major Deck Type* | | |
| **Roof Membrane Type** | | Steel | Concrete | LIF |
| - Built-up | Membrane | Needs Base | FA [2] | MF/BS |
| | Insulation | LLB | LLB | LLB |
| - Modified Bitumen | Membrane | Needs Base | FA [2] | MF/BS |
| | Insulation | LLB | LLB | LLB |
| - Single-ply/Sheet | Membrane | Needs Base | Needs Base | Needs Base |
| | Insulation | LLB | LLB | LLB |

Key:

| | |
|---|---|
| MF – Mechanically Fastened | LLB – Loose-laid Ballasted |
| FA – Fully Adhered | MF/BS – Mechanically Fastened Base Sheet |
| MF/BS – FA – Mechanically Fastened Base Sheet, Fully Adhered Insulation | Needs Base – Requires base of insulation between deck and roof assembly |
| LIF – Lightweight Insulating Fill | LL – Loose-laid |

| | |
|---|---|
| Note 1: Loose-laid insulation would require a mechanically fastened or loose-laid ballasted membrane to provide uplift resistance, thus, a fully adhered membrane would be appropriate over mechanically fastened insulation only. | |
| Note 2: In a Protected Membrane Assembly, the fully adhered method of attachment may require modification to include an insulation board as a separator between the deck and the membrane in the case of precast concrete deck systems with frequent joints. | |

If the general guidelines in Table 1 are followed, the designer will be led to the choice of attachment method that is compatible with the roof deck, insulation, and membrane system, and provides a commonly accepted attachment method. Although the table is intended as a guideline, the designer should follow common sense and code requirements for wind uplift resistance in choosing attachment methodology.

## 5 MATERIAL DURABILITY

Material durability refers to the roof system's ability to resist the spread of fire, weathering and natural or man-made impact without breaking down. The roof system should be rated for its ability to resist the spread of fires within a building, and the roof surface must be resistant to the spread of fire.

Membrane type, thickness and surface treatment have a significant effect on fire resistance and roof system durability. Not only must the roof membrane and surfacing have the ability to resist the spread of fire, the membrane must have the capability to expand and contract to prevent splits and tears from extreme temperature changes. Those capabilities should be present when a roof is constructed, and they should remain at adequate levels throughout its life cycle. Thus, in order for a roof system to perform adequately, it must have adequate material durability when it's constructed, and not have a tendency to lose those characteristics as it ages.

For the most part, roof membrane materials today posses the material durability to resist weathering and building-related movement, and most have means of meeting today's stringent fire codes. What some designers fail to realize is that when a material meant to weather in one specific climatic condition is used in a totally different environment, it may not be suitable for the conditions of the new environment. A relatively thin, single-ply membrane that is suitable for a sunny, mostly dry climate in the southwestern United States may be totally inadequate for a severe-winter northern U.S. environment where roof traffic is common. A particular membrane assembly that is used in environments where insulation is not needed or used may not be appropriate in a heavily insulated assembly. Many manufacturers have multiple membrane products for numerous surfacing configurations. Often, the manufacturer will have similar membrane products, some of which are rated for surface burning characteristics, others of which are not. The designer needs to be ever vigilant to specify the correct products.

The roof designer is best advised to carefully research the prior use of the chosen roof system product and demand a solid track record of performance over many years. If this is not done, the owner should be advised of the risk of using materials and systems that do not have a proven track record in the climate required.

Data on the service life of various roof systems may well be the best indicator of material durability. It is generally accepted that the service life of single-ply membranes is growing with improvements in seaming and flashing technology (Hoff 1997). The durability of the single-ply system in the past may have been judged more by how many leaks developed over time, as opposed to how well the membrane material sustained weathering. A case in point is the longevity of some single-ply roof membranes. Although many single-ply membranes offer good waterproofing protection over an extended period of time, the documented service life of some single-ply membranes through the late 1990's was less than more traditional bituminous membranes (Cash 1997). This may have been due in part to its relative lack of material durability under foot traffic that caused punctures and leaks.

Roof system traffic will generally have a significant effect on the service life of a roof system (Kyle *et al.* 1997). There have been several attempts in the United States roofing industry to standardize roof membrane impact testing as it relates to both roof traffic and environmental influences (hail). These attempts are often met with resistance due to the complexity and sheer number of membrane types and surfacing offered in the market. Currently, only some of the major roof membrane material standards in the United States have some sort of impact resistance requirements. Other countries have had some success with the rating of various roof systems by their resistance to movement, weathering, and puncture. These material durability-related issues have been documented, tested, and are incorporated into a rating system used in France since 1989 (Chaize *et al.* 1991).

Thin, unreinforced membranes are not likely to have the same resistance to roof traffic as a multiple-layer, built-up membrane product. Even reinforced single-ply products may be more susceptible to cuts, tears, and puncturing than a multi-layer, built-up membrane. The roof designer should consider the amount of serviceable equipment on the roof before deciding on an appropriate membrane and surfacing. The durability of the roof surface is a key factor in achieving longer service life. Table 2 is the author's general rating system of the durability of a roof surface relative to roof traffic.

**Table 2. Resistance to Roof Traffic**

| Roof System | Surfacing | Traffic Resistance |
|---|---|---|
| | | |
| Built-up | Gravel | High |
| (3 or 4-ply) | Smooth Surfaced | Moderate to High |
| | | |
| Modified Bitumen | Gravel | High |
| (2-ply) | Granular Surface | Moderate to High |
| | Smooth Surfaced | Moderate to High |
| | | |
| Single-ply | Unsurfaced/Exposed Membrane | Low to Moderate |
| | Ballasted | Low to Moderate |
| | | |
| Protected Membrane | Ballasted Insulation (Membrane beneath) | High |

Ultimately, the roof designer is responsible for selection of a roof system that meets all of the durability requirements. These durability requirements may be code or standard-mandated such as: fire resistance, puncture or tear resistance, elongation, and weatherability. Other durability requirements may be imposed by the climate in which the roof system is intended to serve, and by the traffic the roof system may be required to endure. Although these requirements are rarely code-mandated, the designer should pay as close attention to those requirements as to those that are code-mandated.

## 6 CONSTRUCTABILITY

Constructability refers to the construction factors involved in placing a roof system on a particular building, taking into account building location, height, type, use, and occupancy. The design process should take constructability into account in choosing methods such as hot asphalt application, cold adhesives, fasteners, ballast, roof access, and numerous other factors.

Building location is often a factor in the choice of roof system. Certain materials may have the advantage of being readily available while others may not. Asphalt-based products may be quite expensive in some areas while readily available and inexpensive in others. Fluctuations in the cost of crude oil may affect the cost of certain insulation and membrane types. Recently in the United States, energy cost has impacted certain high-energy use production facilities, thus affecting the cost of the final product.

Building use is a key factor in determining the constructability of replacement roof systems, new construction, and in additions adjacent to occupied structures. Hospitals and health care facilities may require more occupant-friendly material that avoid the use of hot asphalt and the associated fumes and fire safety issues. Buildings located in busy urban areas may have access problems that require a roof system that does not depend on full-time access to the roof. Crane access may be limited, and thus limit the amount of material transport that can occur. In these instances, roofs requiring hot asphalt from kettles and tankers may not be feasible.

Building height can limit the use of hot asphalt in areas where kettles are not permitted on rooftops. Fire safety regulations often prohibit the use of asphalt kettles on the rooftop and require that all kettles and tankers be kept a minimum of 25 feet from the building. This makes the use of hot asphalt difficult for building heights over about 80 feet. More powerful pumps and insulated piping can make ground-based kettles more effective for taller structures; however, additional expense is involved.

Torch-applied asphalt-based products overcome some of these access issues; however, wood-framed, and wood deck structures are somewhat more risky candidates for torch-applied products when considering the risk of fire during construction. Torch-applied modified bitumen products are usually excellent candidates for protected membrane assemblies over concrete decks. Torch-applied modified bitumen membranes can offer some access advantages in dense urban applications since roll materials are transported to and stored on the roof.

When used with cold-applied adhesives, modified bitumen products usually offer better durability than single-ply membranes, without the construction issues of heating bulk asphalt for built-up assemblies. Cold-applied adhesives have increased in popularity and effectiveness over the last several years. These products have enhanced the use of traditional asphalt-based modified bitumen products without the reliance on heating.

The designer should also consider durability of the membrane during construction. Roof construction is a rough and considerably damaging process. Flashing and sheet metal construction, and follow-on rooftop mechanical work is often the most damaging phase of roof construction; often more damaging than years of service life. The durability of the membrane product to construction traffic is an important consideration. Again, the more durable membrane products will have the advantage in resisting construction traffic.

Aside from building location, use, height, and other physical factors, roof deck and interior construction has a significant impact on constructability. Attachment methods will impact constructability, as outlined in an earlier section of this paper. For example, the presence of a non-nailable deck on a tall building in a dense urban environment may lead the designer to choose a ballasted single-ply membrane or a modified bitumen system (torch applied) in a Protected Membrane Assembly. The choice between these two systems would then become one of durability over price, with some potential difference in expected service life. However, both types of systems can be appropriate. On the other hand, a mechanically fastened system would not likely be appropriate from a constructability and attachment standpoint.

Steel or lightweight insulating fill decks are not always candidates for ballasted roof systems since the weight addition is significantly larger than traditional gravel or smooth surfaced asphalt-adhered systems. Concrete decks generally have higher dead load capacities; however, all systems should be checked before adding ballast to a roof system.

The interior conditions and finish beneath the deck are also important constructability issues. The presence of spray-applied fireproofing on the underside of a steel deck needs to be considered when re-roofing with a mechanically fastened system. Fastener penetration through fireproofing material can lead to delamination in loosely adhered areas, and compromising of the fireproofing system. The presence of asbestos in older fireproofing systems may also limit the use of fasteners through a steel deck. Construction vibration through any type of deck system must also be considered in and around adjacent fireproofed building systems. The presence of high-heat areas, refrigerated spaces, high interior humidity, clean rooms, and secure areas are just a few of the other design factors that have to be considered in design of the roof system.

Steel deck fasteners through to an occupied space where the underside serves as the finished interior surface can be an issue for re-roofing. This is an issue when replacing an older built-up roof that was constructed when hot mopping insulation to a steel deck was acceptable. A practical method for roof replacement in these cases is mechanical attachment through the steel deck. However, the presence of the protruding fasteners on the underside can be problematic, especially where visible in gymnasiums, atriums, and where the deck underside is painted.

The designer should consider constructability factors when choosing a new or replacement roof system. How the roof system interacts with all of the building's other systems is important, as well as how easily the roof system is constructed. The designer is also well advised to determine how well the specified system matches with the expertise of the contractors that will likely be performing the construction.

## 7 MAINTENANCE

Maintenance is required for all roof systems. Properly maintained roof systems will have longer service lives and reduced ownership costs. Maintenance practices will generally not vary significantly by roof type; however, some roof types are considered more easily maintainable than others. Traditional gravel and granular surfaced built-up and modified bitumen membrane systems are easily maintained because surface damage to the membrane is visible and most often obvious. In the case of an exposed, coated, gravel, or granular surfaced membrane, routine inspection of the membrane should be performed. This also applies to exposed-membrane single-ply roof systems (fully adhered and mechanically fastened). Membrane damage can be visually detected and repaired.

Ballasted roof membranes and Protected Membrane Assembly membranes are not nearly as visible, and many consider these membranes to not be maintainable. It is unlikely that any routine inspection process should or would require ballast removal or insulation and ballast removal in order to carry out the inspection process. This process is likely to do more harm than good by damaging the membrane during the ballast removal and replacement process.

Even though all membrane and covering systems are not created equal with respect to inspection and maintenance, most of the other roof system components are as equally maintainable regardless of roof type. Most roofs have exposed flashing and drainage systems that require periodic inspection and cleaning. Metal flashing systems are common to most types of roof systems, even though metal type will vary. Most roof systems also contain flexible sealants, expansion joints, and penetration flashings that can be monitored and maintained. All roof drains, scuppers, collector heads and gutters should be inspected and cleaned at least two times per year, or more often when wind-born debris and leaves are a factor.

Roof system coatings such as aluminized and acrylic-based coatings are often the most maintenance-intensive portions of a roof system. Coating system longevity is often much shorter than expected roof life cycle and will require periodic re-application to be effective.

The important factor about maintenance is that the designers recognize which portions of the roof system will require maintenance and alert the Owner as to which systems require maintenance and to what level of detail. Maintenance may also include periodic leak repair. A difficult or non-maintainable membrane such as a ballasted or protected membrane assembly should require a membrane type that is high on the durability scale in order to reduce the demand for leak repairs. If it is not, an Owner may spend an inordinate amount of time moving ballast or uncovering protected membrane assemblies to chase leaks. Thus, a roof's "maintainability" becomes an important design factor.

## 8 SUMMARY

If designers were to pay closer attention to these six key factors, the average service life of a roof system could be extended significantly. This paper outlines these six key design factors and their effect on the service life of a roof system. Using these design concepts adds little to the construction or replacement cost of a roof system, but can have a significant positive effect on service life and will reduce life cycle cost. A summary of the key factors is listed below.

1. **Slope** – Provide a minimum of ¼ inch per foot slope for low-slope roof applications.
2. **Drainage** – Provide at least the minimum drainage and overflow protection as required by code, and err on the side of more drainage than required when designing roof systems for small areas, multiple levels, and unusual shapes.
3. **Attachment** – Choose attachment methods carefully, and utilize proven, code-required, tested attachment methods matched to the deck system.
4. **Durability** – Make roof system durability an important design factor and use life-cycle to help the Owner choose the most cost-effective roof system.
5. **Constructability** – Each roof type has advantages and disadvantages when it comes to constructability; choose a system that can be built.
6. **Maintenance** – All roof systems require similar maintenance, however, all membranes are not created equal; if the roof system is to be covered, make durability and longevity of the membrane a high priority.

Designers should remember that all roof systems are not equal, and each building has different requirements. There is no single roof system that is applicable in all situations. It is not the choice of material or specific manufacturer that will make a roof system work, it is a combination of all of these design factors taken in total that will provide longer-lasting roof systems.

Although these are not the only factors a designer may encounter, they provide the basis for sound design judgment and will lead to more durable roof systems with longer service life. The durable roof of the future will utilize technology to design for slope and improved drainage, better adhesives and fasteners for solid attachment to meet wind uplift requirements under a variety of deck conditions, more durable, thicker materials to stand up to weathering and ultraviolet radiation, worker and occupant-friendly materials to enhance constructability, and design with maintenance of the total roof system in mind.

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# Surface Roughness Slip Resistance And Relationship To Floor Cleaning


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**Summary: Determination of slip resistance of flooring materials is of importance to both designers and specifiers when responding to client requirements, and to building owners, users and facility managers when buildings are commissioned and over the lifetime of the floor. Clearly, acceptable slip resistance of floorings is a significant safety requirement of any building with an industrial, or public service operation. Assessment can be made as a routine part of production, to ensure client satisfaction, or upon the floor in-service, to ensure the required slip resistance has been maintained during routine service life.**

**Slip resistance is commonly measured in the UK using the TRRL pendulum tester but the use of surface roughness measurement to assess slip resistance is growing in popularity. This paper identifies a relationship between the surface roughness of flooring materials and the cleaning properties by presenting two case studies. The BRE studies examined in-service floor tiles that were retaining dirt, irrespective of the cleaning regime, or the traffic. The relationship between surface roughness and dirt retention enabled the identification, in one case, of areas where the cleaning regime was insufficient.**

**Examination of the data produced from the two case studies suggests that the peak to valley distances of the sub-millimetre surface profile and the widths of the peak to valley elements are important parameters influencing dirt retention. These parameters can be measured using one of the instruments recommended in the Guidelines of the UK Slip Resistance Group (2000), although the measurement of the mean width peak to valley distances is not required when assessing slip resistance.**

**Thus, in addition to providing information concerning slip resistance, the assessment of sub-millimetre surface profile characteristics may also be advantageous in quantifying cleaning properties, and provide valuable information both before installation and during service life.**



**Keywords: Ceramic, terrazzo, slip, surface roughness, cleaning**


## 1 INTRODUCTION

Determination of floor slip resistance is of importance to ensure the appropriate properties will be obtained for new floors or floors maintained in-service. Measurement of floor slip resistance is most commonly conducted in the UK using the TRRL pendulum tester, although the use of surface roughness measurements is growing in popularity. Recent studies at BRE have indicated that a relationship exists between the sub-millimetre surface profile characteristics and the cleaning properties of flooring materials. Thus, the movement towards roughness measurements to assess slip resistance may prove to have additional benefits in providing characterisation of the floor cleaning requirements. Although there is still much work to be completed to quantify this relationship this paper presents two case studies where a direct correlation between the sub-millimetre floor surface profile and the cleaning properties has been established. It is hoped that these data will assist others conducting research in this field.

## 2 MEASUREMENT OF FLOOR SLIP RESISTANCE

In the UK the pendulum tester is commonly used to assess the slip resistance of floorings both in-situ and in the laboratory. The assessment technique is discussed in guidance from the UK Slip Resistance Group (UKSRG, 2000), the BRE Information

Paper 10/00 (Yates and Richardson, 2000) and the CIRIA Report 184 (Gatfield, 1998). This method is currently under consideration as a European Standard (prEN 13036-4: 1997) to assess skid resistance. The pendulum tester relies on the measurement of the coefficient of friction between a rubber slider and the flooring to assess the resistance to slip. For assessment of flooring a Four S rubber slider is recommended for most floors and the results are categorised according to Table 1.

**Table 1. Categories of slip resistance**

| *Potential for slip* | *Four S pendulum value* | *Rz surface roughness (mm)* |
|---|---|---|
| High | 25 and below | Below 10 |
| Moderate | 25 to 35 | Between 10 and 20 |
| Low | 35 to 65 | Above 20 and up to 30 |
| Extremely low | Above 65 | Above 30 |

The recently published Guidelines from the UKSRG (2000) support the use of surface roughness measurements to assess slip resistance, by providing test methods for both on-site and laboratory testing. Although not recommended as a replacement for the pendulum test the surface roughness is considered important, particularly where water contamination is likely. Equally, the UK Health and Safety Executive (1997) recognise that both the friction between the floor and the shoe and the sharpness of the granular micro-surface peaks influence slip resistance.

## 3 MEASUREMENT OF SURFACE PROFILE CHARACTERISTICS

The Guidelines of the UKSRG (2000) recommend the use of the Taylor Hobson Surtronic 3+ or Duo roughness meter, to establish the slip resistance of the floor in-situ or in the laboratory. Surface profile is quantified by parameters that relate to certain characteristics of the surface, defined in BS ISO 4287: 1997. The parameters discussed in this paper include Ra, Rq, Rc and RSm. These parameters can be classified in three groups, according to the type of characteristic that they measure:

- Amplitude parameters: Are measures of the vertical displacements of the profile, essentially peak and valley measurements, for example Ra, Rq, Rc.
- Spacing parameters: Are measures of spacings of profile elements (the surface profile between a peak and adjacent valley), for example RSm.
- Hybrid parameters: These relate to both the amplitude and spacing of the surface profile elements.

### 3.1 Definition of surface profile parameters

The Mean Line is commonly used in surface profile characterisation and it is derived by the least squares method. In basic terms it is a line which bisects the profile such that the area above it and below it are equal and a minimum, as shown in Fig. 1.

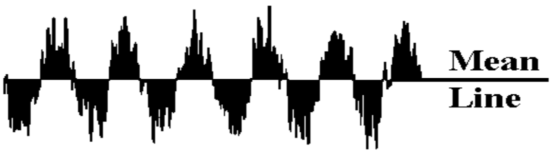


**Figure 1. Mean line of a surface profile**

The profile is described as a series of heights above or depths below the mean line, Z ordinates, at a distance along the sampling length, x ordinates, the ordinate value is written as Z(x). This is shown schematically in Fig. 2.

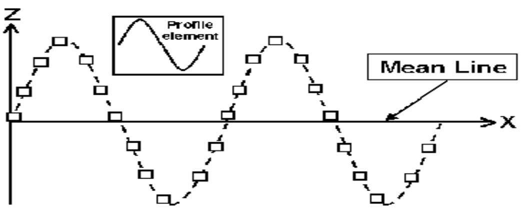


**Figure 2. Z(x) ordinates of a surface profile**

Ra is the average roughness of the surface and is the arithmetical mean of all the Z(x) values. Rq is the root mean square of these measurements, and thus is independent of the sign of the roughness (whether it is a peak or a valley) thus Rq is usually larger than Ra for the same sample set.

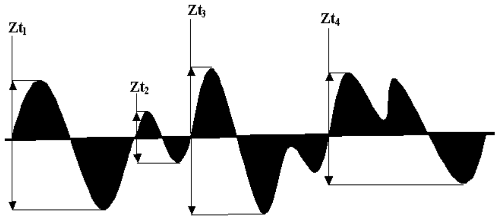


**Figure 3. Schematic diagram of heights of profile elements ($Zt_n$)**

Rz is the roughness parameter used in the UK Guidelines (UKSRG, 2000) to assess slip resistance, the threshold values are given in Table 1. This Rz value corresponds to Rc in BS ISO 4287:1997 and is the mean value of the profile element heights (Zt) within the sampling length as shown in Fig. 3.

Also discussed in this in this paper is RSm, the mean value of the profile element widths (Xs) within a sampling length (Fig.4).

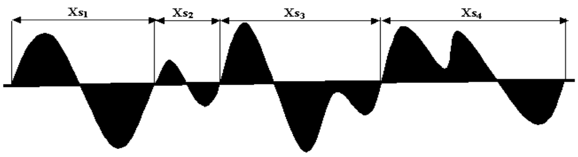


**Figure 4. Schematic diagram of Xs**

## 4 CLEANING PROPERTIES - CASE STUDIES

### 4.1 Case study 1 - Ceramic tiled floor

BRE were asked to examine the darkening of a ceramic tiled floor in a catering facility. BRE undertook an extensive study of the tile surface characteristics using optical and scanning electron microscopy (SEM) and laser profilometry in order to determine the causes of the darkening.

Optical and scanning electron microscopy of the tiles confirmed that the darkening was caused by dirt retention at the surface of the tiles, Figs. 5 to 8.

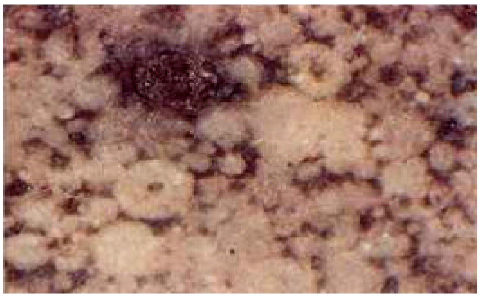

**Figure 5. A severely darkened tile, optical microscope image X 150**

**Figure 6. A severely darkened tile SEM image, scale bar is 1000 mm**

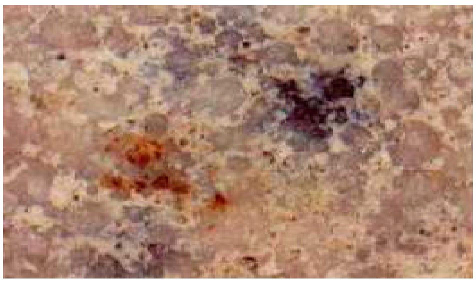

**Figure 7. An unaffected tile, optical microscope image X 150**

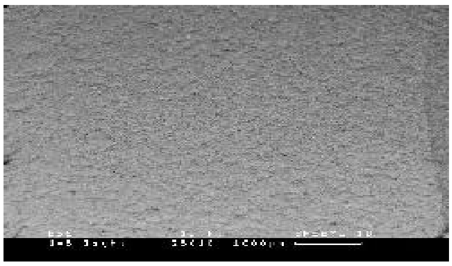

**Figure 8. An unaffected tile SEM image, scale bar is 1000 mm**

The optical micrograph (Fig. 5) shows the presence of the dirt at the surface of the darkened tile, which is absent in the unaffected tile (Fig. 7). This is confirmed as dirt of high carbon content by the dark appearance of this material in the back scattered electron SEM image (Fig. 6). Under the SEM lighter elements appear darker, thus carbon will appear darker than the surrounding alumino-silicate tile substrate. Few carbon-containing areas are visible on the unaffected tile (Fig. 8).

Surface profile was assessed for a number of tiles removed from different locations of the floor, subject to different cleaning regimes and traffic. The tile surface profiles were assessed using a Cyberoptics Microscan 3D laser profilometer. Measurements were taken at 10 different locations on each sample tile and then averaged to represent each tile. Surface scans were undertaken in 10 μm steps, each scan compromising 1000 steps, producing a sampling length of 10 mm. In this case study the samples were assessed for average Ra and Rq, given in Table 2.

No relationship was found between the cleaning regime in place and the darkening of the tiles (by visual assessment), the darkening being indicative of the quantity of dirt retained by the tile. These data are shown in Table 2, where 1 is the most rigorous cleaning regime and 6 the least rigorous (no pro-active cleaning under a disused conveyor belt). Table 2 also contains an assessment of the traffic the floor was subjected to at these points, where 1 is the heaviest traffic and 4 the lightest (no traffic under a disused conveyor belt). No direct relationship can be seen between the darkening and the traffic.

**Table 2. Cleaning, traffic and roughness data for sample tiles**

| *Area* | *Visual description* | *Cleaning regime* | *Traffic* | *Ra (mm)* | *Rq (mm)* |
|---|---|---|---|---|---|
| 7 | Darkened | 1 | 2 | 14 | 15 |
| 7 | Unaffected | 1 | 2 | 11 | 12 |
| 6 | Little Darkening | 2 | 1 | 14 | 15 |
| 6 | Unaffected | 2 | 2 | 11 | 11 |
| 6 | Unaffected | 2 | 2 | 11 | 11 |
| 3 | Unaffected | 3 | 3 | 12 | 12 |
| 3 | Unaffected | 3 | 2 | 12 | 13 |
| 5 | Darkened | 3 | 3 | 12 | 12 |
| 5 | Darkened | 3 | 3 | 12 | 12 |
| 8 | Severely Darkened | 4 | 1 | 22 | 25 |
| 8 | Severely Darkened | 4 | 2 | 17 | 19 |
| 1 | Little Darkening | 5 | 2 | 16 | 17 |
| 1 | Little Darkening | 5 | 3 | 13 | 14 |
| 1 | Unaffected | 5 | 3 | 13 | 13 |
| 1 | Unaffected | 5 | 3 | 12 | 13 |
| 1 | Unaffected | 5 | 3 | 12 | 14 |
| 2 | Darkened | 5 | 3 | 16 | 18 |
| 4 | Unaffected | 6 | 4 | 12 | 13 |
| 4 | Unaffected | 6 | 4 | 12 | 13 |

The relationship between the tile darkening due to dirt retention and the Ra and Rq values are given in Figs. 9 and 10. The graphs also show the $\pm$ 2 standard deviation values taken from the average of the 10 assessments on each sample tile. These error bars show that there is some overlap in the 95% confidence limits of the average measurements of the tiles retaining dirt and the unaffected tiles. This overlap does not affect the conclusions drawn from this case study but indicates that further data collection and analysis would be required before specification of a threshold roughness related to dirt retention could be established.

Figures 9 and 10 demonstrate the relationship between the roughness of the tile surface and dirt retention. In all but two examples is the dirt retention accompanied by higher surface roughness of the tiles. For these two exceptions, it is likely that the cleaning regime (Number 3) is not sufficiently rigorous, or that is it not conducted adequately. This example demonstrates that the use of surface roughness measurement will differentiate between causes of dirt retention.

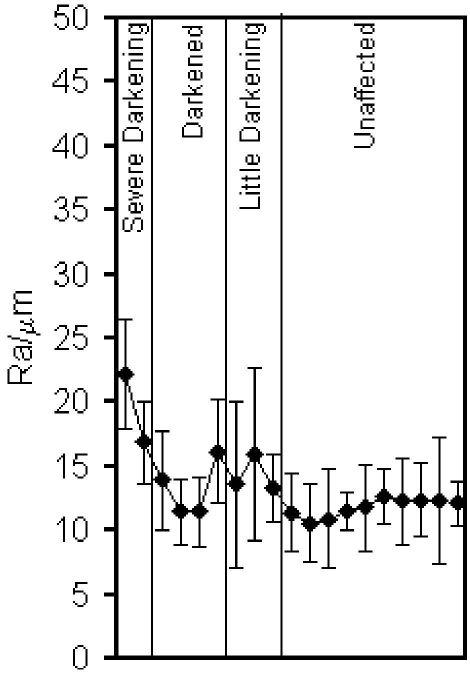


Figure 9. Ra vs visual assessment

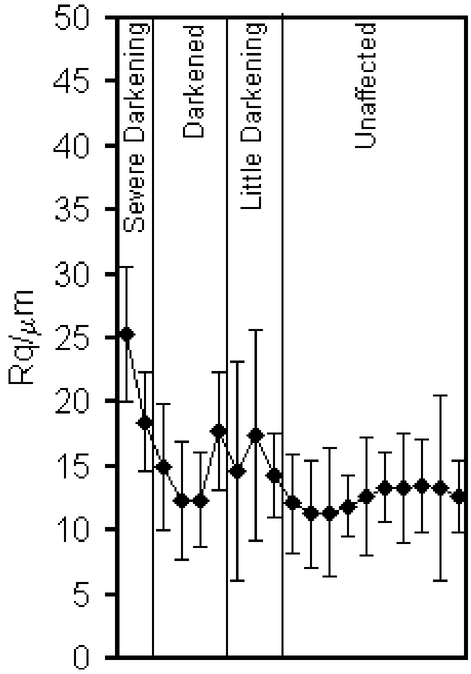


Figure 10. Rq vs visual assessment

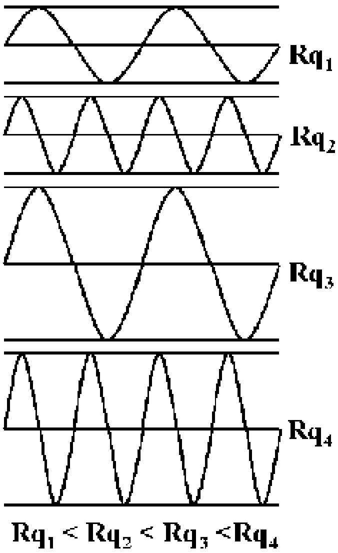


**Figure 11. Impact of increasing Zt and reducing Xt on Rq**

As the Rq values of the tiles retaining dirt are higher than those of the unaffected tiles it can be deduced that either the distance between the peaks and valleys (Zt, Fig. 3) has increased or the width of the profile elements has decreased (Xs, Fig.4), or both. This deduction is shown schematically in Figure 11, although in all cases the arithmetical mean (Ra) would remain identical.

### 4.2 Case study 2 - Terrazzo tiled floor

BRE were commissioned to investigate the dirt retention in the terrazzo tiled floor of a covered shopping mall. The areas that were retaining dirt appeared rougher (visually) than the other areas of the floor. A Taylor Hobson Surtronic 3+ was used to assess the mean roughness values Ra and Rq, the mean peak to valley distance Rc (Fig. 3), and the mean width of the profile elements RSm (Fig. 4). Rc measurements were taken although there was no implication that the slip resistance of the floor was inappropriate for the situation.

The surface measurement was conducted in-situ upon tiles retaining dirt and those that were in routine service but presented no difficulty in cleaning. The cleaning regime in all these areas was considered to be identical. The average of the data collected (10 measurements per tile) for 5 tiles, is presented in Table 3, along with a visual description of the tile surface.

**Table 3. Data for terrazzo tiles**

| *Tile Number* | *Visual Assessment* | | *Ra (mm)* | *Rq (mm)* | *Rc (mm)* | *RSm (mm)* |
|---|---|---|---|---|---|---|
| | *Cleanliness* | *Surface* | | | | |
| 1 | Dirty | Rough | 5.7 | 7.8 | 31.3 | 495.5 |
| 2 | Clean | Smooth | 1.7 | 2.3 | 12.1 | 207.5 |
| 3 | Dirty | Rough | 6.3 | 8.9 | 30.6 | 516.7 |
| 4 | Clean | Smooth | 1.7 | 2.4 | 11.0 | 226.9 |
| 5 | Dirty in Places | Partially Rough | 5.7 | 7.8 | 29.2 | 514.2 |

The data in Table 3 confirms that all the tiles have at least moderate slip resistance (categories given in Table 1) and that the tiles that appear visually rough and that are retaining dirt are quantifiably rougher than those that appear smooth and are unaffected by dirt retention.

Taken collectively, the results presented in Table 3 indicate that, on average, the peaks to valley distances are larger (Rc is larger) and at further widths (RSm is larger) for the rough tiles that retain dirt, compared to the smooth tiles which are easy to clean. This is represented in Fig. 12.

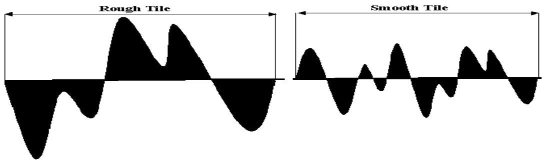


**Figure 12. Schematic diagram of rough and smooth tiles in shopping mall**

Figure 11 demonstrates that a decrease in the mean width of profile element (RSm) or an increase the peak to valley distance (Rc) will result in a increase the root mean square roughness (Rq). Equally an increase in RSm or a decrease in Rc will result in a reduction in Rq. Thus the combination of the increased Rc and increased RSm, results in smaller changes in Rq.

Hence the difference between the Rq for the rough, dirt retaining tiles and the smooth, unaffected tiles (Table 3), although apparent, is less obvious than the differences between the Rc and RSm. This suggests that the measurement of Rc and RSm may be more appropriate when characterising the cleaning properties of surfaces.

Figures 13 and 14 contain the Rc and RSm averages, with 95% confidence limits ($\pm$ 2 standard deviations), plotted against the visual assessment of roughness, indicative of dirt retention. As with the previous case study there is some overlap in these confidence limits, again indicating that more data is required before threshold roughness values for Rc and RSm related to dirt retention can be established. The large 95% confidence limits associated with the partially rough tile result from the variation in the in roughness across the tile surface.

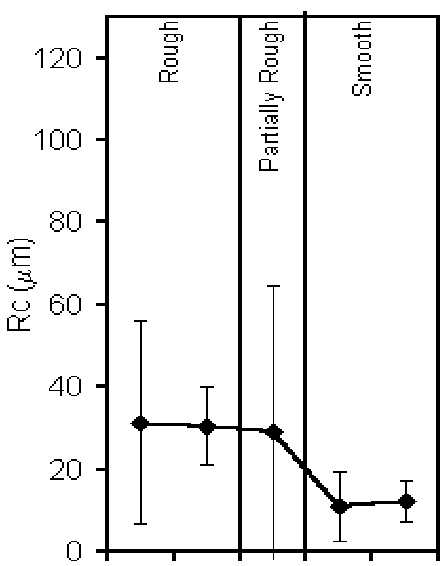


**Figure 13. Rc vs visual assessment**

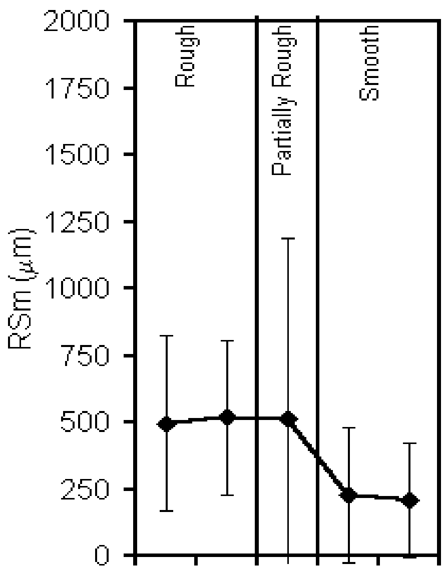


**Figure 14. RSm vs visual assessment**

## 5 CONCLUSIONS

The relationship between sub-millimetre surface profile and slip resistance is recognised as important and measurement of surface roughness recommended as a tool to assess slip resistance in areas subject to water contamination. The case studies presented here indicate a relationship between the surface profile and dirt retention although there is a requirement for more data before this relationship can be fully quantified. However, the evidence from the studies presented here suggests that the relationship between the peak to valley distances (Zt, Rc) and the width of the peak to valley profile elements (Xs, RSm) are both of importance in dirt retention.

Thus, in addition to providing information concerning slip resistance, the assessment of surface profile characteristics, may also be advantageous in quantifying cleaning properties, and provide valuable information both before installation and during service life.

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# Colour Specification As Part Of The Construction Process

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**Summary: Too many construction projects undergo remedial action because the client is unhappy with the colour, or the colour matching, of the final constructed product. This is especially common with tiled floors, coloured concrete and stone and terracotta cladding. Difficulties arise for clients, designers, specifiers and contractors when matching batches of materials on site in differing light conditions. Colour specification and testing is a powerful tool to overcome all these problems.**

**Colour measurement can be used in two ways:**

1. **To specify and agree the product colour, and acceptable tolerances, with the client.**

2. **To provide an understanding of the degree of variability to be expected between batches of the product and hence determine the degree of mixing of batches required on site, to establish a randomized effect.**

**This paper provides an overview of the colour measurement process, practical and usable by architects, designers and specifiers. Colour measurement is explained through discussion of two colour measurement systems (L* a* b* and L* C* h*) and by an exemplifying case study.**

**Although there are standards that specify the degree of acceptable colour difference in some construction products, many of these standards are only applicable to solid coloured materials, not materials with a speckled appearance, such as many floor tiles, or natural stones. The data collected in the case study is analysed using statistical methods to establish levels of colour difference. The data and subsequent analysis has been used to suggest a maximum acceptable colour difference ($DE_{cmc}$) of 1.02 units, colour matching tiles construction products with a speckled appearance (made up of coloured dots between 2 mm and 5mm).**

**Keywords: Colour, specification, tolerances, ceramics, $DE_{cmc}$**

## 1 INTRODUCTION

Colour measurement and colour matching are techniques that have been developed and used in the textile, paint and tableware industries for many years. The systems are used to ensure that products are as specified and consistent between batches. Given the requirements of these industries, the instrumentation to assess colour has become more easily available and more robust.

However, these techniques are infrequently used within the construction industry, and when they are employed it is often to confirm the differences alleged by the client rather than to specify the colour and the acceptable tolerances of construction materials.

### 1.1 Colour perception

The perception of colour is dependent on several factors unrelated to the true colour of the product. Every individual will perceive colours differently, whether considered colour blind or not, and the colour perceived will be dependent on the lighting. These perceptions are exemplified by two everyday occurrences:

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1) Most people have watched television at friend or relatives house and thought the picture is too red or too green. However, the friend or relative will have set the television picture to show colours as they see them.

2) Many people have purchased what they believe are matching, or complimenting colours of clothes, only to find that outside the shop in the daylight or at home under tungsten lighting they are not the same.

Thus the colour vision of the individual and the lighting environment where colours are viewed will affect the choice of colours for construction products, and whether they are then considered acceptable when installed. This situation is compounded as coloured products are often produced in batches. Without colour measurement or specification of the required colour, how can it be accurately established that batches are the same colour, especially if this judgement is based on the colour vision of the buyer and the lighting environment the batches are viewed in?

## 2 COLOUR MEASUREMENT

There are several different systems for defining and measuring colour, but the most common are the L* C* h* system and the L* a* b* system. L* C* h* uses polar co-ordinates to define colour, L* a* b* uses Cartesian co-ordinates. Both systems are equally useful, although the polar system is more robust when examining differences in colour. For the L* C* h* system:

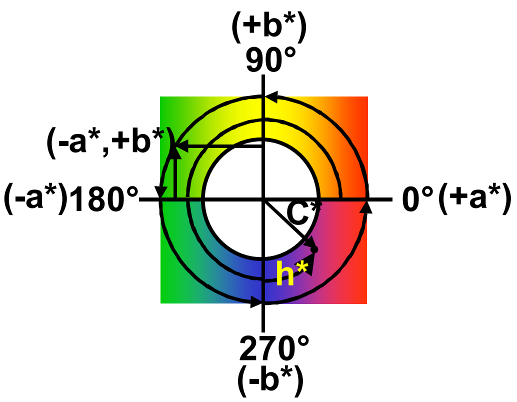
- L* is the lightness of the colour, 0 being black and 100 being white,
- C* is the chroma, effectively the quantity of the colour present;
- h* is the hue angle, expressed as a number of degree rotation, defining the colour present.

Thus for a pink colour, h* defines the colour present, the underlying red tone, C* the quantity of the colour present, that is, how much red is there, and L* defines the lightness, such that it is light and appears pink. The L* a* b* system is based on similar principles, larger numbers, positive or negative, indicate an increased presence of a colour. For the L* a* b* system.

- L* is the lightness (high numbers) and darkness (closer to zero) of a sample.
- a* is the amount of red (positive numbers) or green (negative numbers) in a sample.
- b* is the amount of yellow (positive numbers) or blue (negative numbers) in a sample.

Thus for a pink colour, a* will be positive and b* will define the type of red colour, whether it is scarlet (+b*) or a crimson (-b*), and L* again defines the lightness, such that it is light and appears pink.

Figure 1 is a simplified schematic diagram of the C* h* and the a* b* co-ordinates in colour measurement. Figure 2 demonstrates the L* quantity, equivalent in both systems.



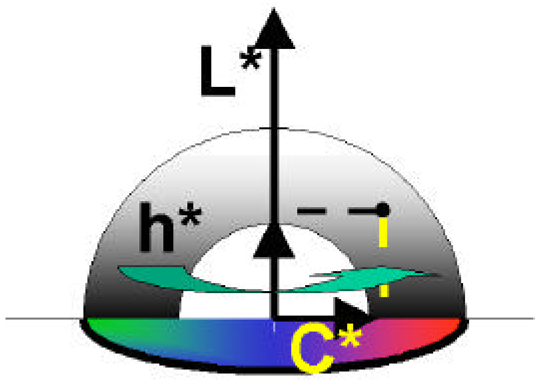




**Figure 1. Schematic Diagram of the C*, h* and a*, b* colour values Measuring differences in colour**

**Figure 2. Schematic Diagram of L* colour values**

Using the same system, differences in colour can be assessed by simply subtracting the co-ordinates, to see the movement in 3D space. There are methods of refining changes in L* C* h* and L* a* b* to one numeric value to provide an easy mechanism to assess colour quality in a pass/fail system. Examples of single numbers to define colour change are ΔE and $DE_{cmc}$. Equations concerning the calculation of these parameters can be found in BS EN ISO 105-J03: 1997. ΔE is relatively simple to calculate but does not include any weighting for the relative values of L* C* or h*. $DE_{cmc}$ is more complex to calculate, although more relevant, especially where small colour differences are important. For a novice in the field, it is excellent practice to examine the L* C* h* or L* a* b* variations individually to provide a comprehensible measure of the colour differences.

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## 3 BENEFITS OF COLOUR SPECIFICATION

Acceptable colour differences and tolerances between batches of product are those agreed with the client at the beginning of a construction project, and can be applied irrespective of the locations of the product, say a tile manufacturing plant in Spain, and the customer, an office in central London. This is, in essence, putting the thoughts and agreements made when looking at the samples of tile on a technically sound footing.

This is by far the best way to establish tolerances. For example, ask the manufacturer for a series of typical samples that will show the likely range of colour variation to be expected. Measure these colour variations and agree them with the client. The benefits of doing this are twofold. Firstly, the tile colour and acceptable tolerances has been specified and agreed with the client. Secondly, there will be an understanding of the degree of variability to be expected in the production batches and hence this will influence the degree of mixing of batches required on site, to establish a randomized effect.

However, there are standards that specify the degree of colour difference in some construction products, for example ASTM C609-90 and BS EN ISO 10545-16:2000. Unfortunately many of these standards are only applicable to solid coloured materials, not materials with a speckled appearance, such as many floor tiles, or natural stones. Where a speckled appearance is present it is possible to provide an estimate of the "underlying" colour by measurement of samples at several points over the surface. This will provide the same basic colours and anticipated colour tolerances as previously discussed.

## 4 INSTRUMENTATION TO MEASURE COLOUR

There are many different machines for assessing colours and colour differences. Many are high specification apparatus for laboratory use, but there are also a number of excellent, portable and robust pieces of equipment for on site assessments. Two types of instrumentation are available, colorimeters and spectrophotometers. Colorimeters tend to be lower cost and may be preferred for site work, but spectrophotometers provide more information and are the Building Research Establishment 's (BRE) preferred method of colour measurement. BRE is the UK's leading centre of expertise on building and construction and on the prevention and control of fire.

## 5 CASE STUDY

BRE is regularly involved in construction projects where the assessment of colour deviation during remedial works is involved. One recent assessment involved the detailed inspection of a ceramic tiled floor in a recently constructed kitchen facility. The colour matching of the tiles was considered to be unacceptable by the client, a situation made worse by the poor mixing of batches during construction, leading to blocks of differently coloured tiles being laid. Although there may have been an acceptance of some colour variation, unfortunately, no colour specification had been agreed at the beginning of the contract, and no tolerances were set. An example of the variation seen is shown in Fig. 3. In Fig. 3, of the tiles shown in their entirety, the lower row of tiles appear to be more red in colour than those above.

The tiled floor covered over 15000 $m^2$ of the facility and to retrospectively assess the colour differences required a large data population. BRE took over 1000 colour measurements to generate a statistically significant body of data to represent this floor. In this case the L* a* b* colour measurement system was used. The tiles were of a speckled appearance, made of dots between 2 mm and 5 mm in diameter.

**Figure 3. Examples of unacceptable colour matching of a recently constructed tiled floor Instrumentation**

The samples were measured for colour using a portable spectrophotometer, an X-Rite SP68 Sphere Spectrophotometer. The instrument was calibrated at the start of measuring and at regular intervals during the investigation. The instrument is calibrated for 100% reflection and zero reflection using a high gloss white reference and black trap, respectively. Additionally, such instruments are supplied with colour references which, although not used to calibrate the equipment, can be use to measure instrumental drift, which experience has shown to be very limited. The sample area of the instrument is a circle of 5 mm radius.

### 5.1 Data collection

As blocks of differently coloured tiles were apparent across the floor, colour measurements were made on ten tiles of similar visual colour and the results averaged for the tile block of a particular colour. Additionally, eighteen pristine tiles, never laid, trafficked or cleaned, were sent to BRE for analysis. The pristine tiles were reserved from the original consignment as replacements following breakage and so on. Although the full manufacturing history of these tiles is unknown, it is known they were produced by the same manufacturer and were taken from a selection of the remaining boxes. Hence they were assumed to be representative of the tile consignment. Each tile was measured at ten different locations on its surface and the measurements averaged to provide a colour measurement for the tile.

The average of all these readings for the pristine tiles was used to produce the average pristine tile colour and associated standard deviation, as a benchmark to compare to the in-situ tiles. Thus, the pristine tiles provided the range of tile colours within which the in-situ tiles could be expected to fall with 95% confidence (average ± 2 standard deviations).

If this pristine tile analysis had been conducted in advance of construction the acceptable range of tile colours could have been established to indicate the acceptability of the consignments received on site. In this case study the construction client agreed that the colour variation of the pristine tiles was acceptable, although the colour itself was not acceptable. Thus, the results of this analysis can only be used to judge if the in-situ tiles are of acceptable variability, not if the absolute colours are satisfactory.

### 5.2 Comparison of the tiles in-situ to the pristine tiles

The data were analysed and the results reported numerically and graphically. Figures 4 and 5 show the colours of tile blocks in terms of L*, a* and b*, against the 95% confidence range derived from the pristine tiles.

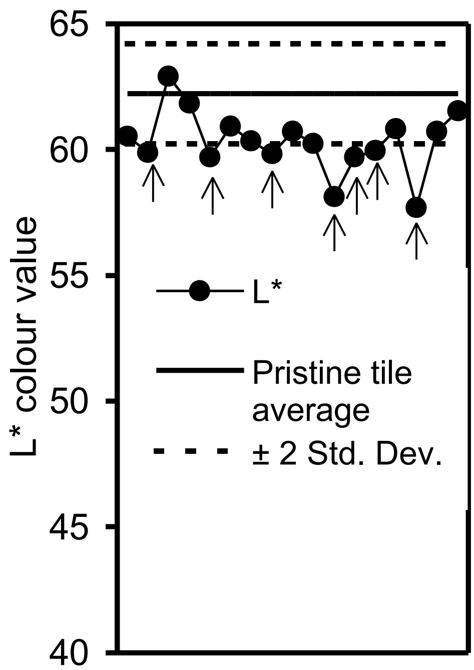



**Figure 4. L* values for in-situ tiles**

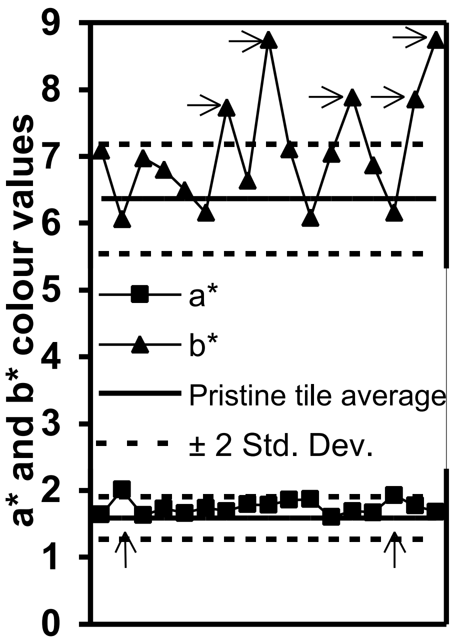



**Figure 5. a* and b* values of in-situ tiles**

The analysis of L* indicates that for 7 out of the 17 blocks of tiles measured the L* value falls outside the 95% confidence limits of the pristine tiles (marked ↑ in Fig. 4). Thus, it would not be acceptable, in terms of colour matching, to lay the pristine tiles next to those tiles falling outside the range. Similar arguments can be made for the a* (marked ↑ in Fig. 5) and b*(marked → in Fig. 5) values measured in comparison to the pristine tiles.

Taken cumulatively, of the 17 in-situ tile blocks measured, 11 fall outside the 95% confidence limits of the pristine tiles. Thus, the pristine tiles could not be colour matched to 110 of the 170 tiles measured. Figures 4 and 5 indicate that there is a significant number of tiles falling outside the range derived from the pristine tiles, in terms of both L* and b* but only one tile fell outside the range in terms of it's a* colour value.

### 5.3 Colour differences caused by service

In order to determine if there had been a change in colour during the operation of the facility, a selection of the tiles were removed from the floor and returned to the laboratory for further analysis. The colour of the tile surfaces was assessed in 10 places over the surface of the tile, as per the pristine tiles. Following this assessment the rear of the tile was ground away to reveal the central potion, which was colour assessed in the same way. This provided a bulk colour measurement to compare to the surface measurement. Similar measurements were made for two pristine tiles, to produce an assessment of the differences between the surface and bulk of the tiles that might be expected irrespective of the service conditioning. These comparisons are given in Figs. 6 and 7.

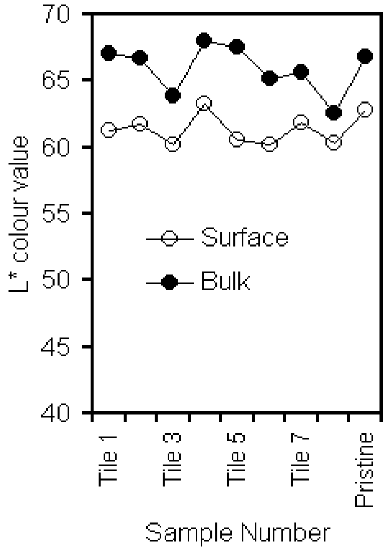



**Figure 6. L* value comparison**

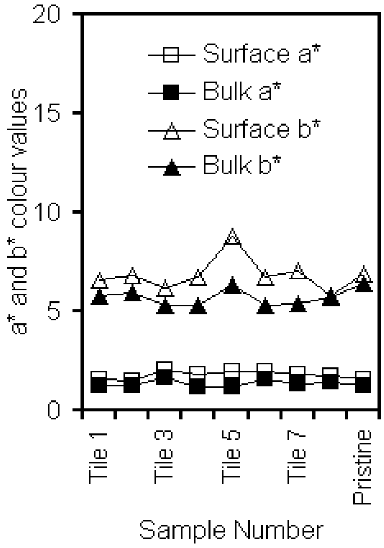



**Figure 7. a* and b* comparisons**

Figures 6 and 7 demonstrate that there is little difference between the bulk and surface colours of the tiles caused by operating conditions at the facility. In all cases the surfaces were darker than the bulk of the tile (lower L* value) and this difference was as expected from consideration of the pristine tile results.

The a* and b* values for the surfaces of the tiles are higher than the bulk values in all cases. Thus there appears to be more colour at the surfaces of the tiles than the bulk. For the a* values the differences appear to be similar to those for the pristine tiles. The difference between the surface and bulk b* values of the tiles removed from the facility are larger than expected when compared to those of the pristine tiles. Thus the surfaces of the tile samples removed from the floor contain more red colour than the bulk. However, it is unlikely that this has been caused by operations at the facility, which would be expected to cause a reduction in colour, and it seems most likely this difference is a relic of the manufacturing process.

This laboratory work supports the conclusion that many of the tiles fall outside the 95% confidence limits derived from the pristine tiles.

### 5.4 Colour differences between the floor tiles

Having identified that the pristine tiles would match the colour the majority of the tiles measured in-situ, further data analysis was conducted to establish the colour tolerances of the in-situ tiles in comparison to each other. Figures 8 and 9 compare the in-situ tile measurements with the average, 68% (± 1 standard deviation) and the 95% confidence limits.

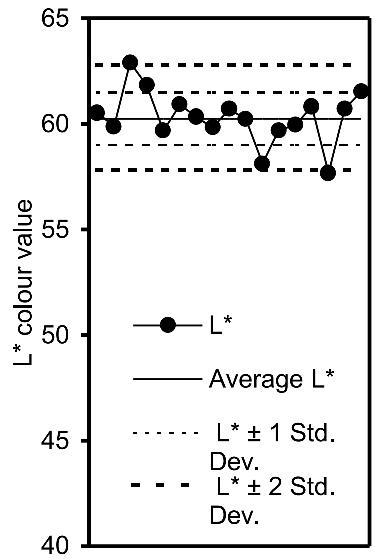



**Figure 8. L* values for in-situ tiles**

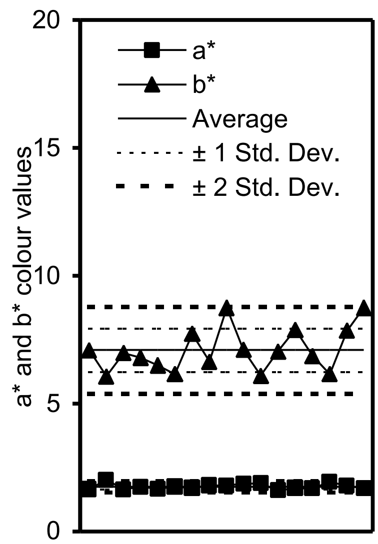



**Figure 9. a* and b* values for in-situ tiles**

Figures 8 and 9 demonstrate that even when analysing the tile colours of the floor against the average colours of this sample set of tiles themselves, a total of 9 tiles fall outside the 68% confidence limits and 3 fall outside the 95% confidence limits. Thus 30 tiles could not be placed, with confidence next to any of the tiles within the sample set, to achieve colour matching.

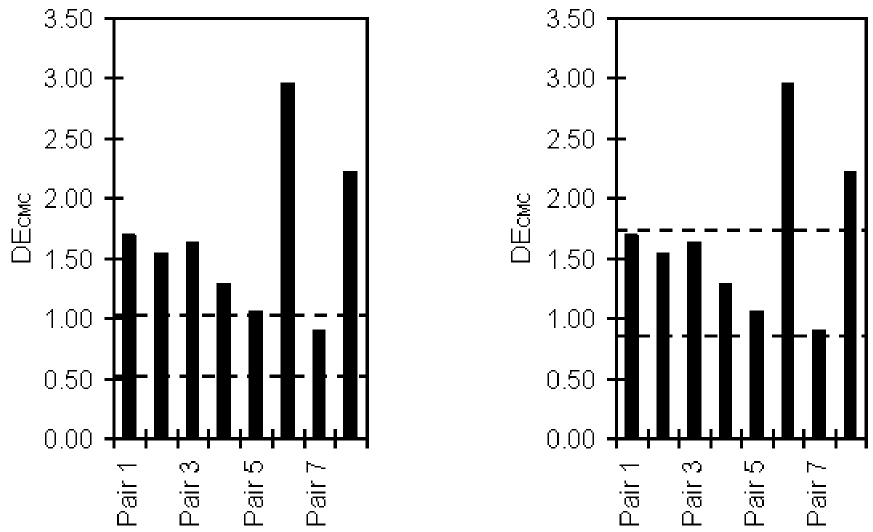
### 5.5 Colour differences related to BS EN ISO 10545-16

This standard has been established to enable the specification of acceptable colour differences between consignments of samples. It uses the calculation of $DE_{cmc}$ to assess these differences and a tolerance of 0.75 units. However, this standard is only applicable to plain coloured glazed ceramic tiles, so would not be applicable to the speckled, unglazed floor tiles considered in this case study.

Using the data gathered from the pristine tiles it possible however to derive an acceptable colour deviation. Table 1 demonstrates the average colour deviation of the pristine tiles from the average colour is 0.51 units, and the maximum deviation, 1.02 units. Thus it is reasonable to assume that for adjacent tiles in the in-situ floor, $DE_{cmc}$ should average 0.51 and be no more than 1.02. The colour deviation of these adjacent tiles, is shown graphically in Figure 10.

**Table 1. Pristine tile colour data**

| *Tile Number* | *L** | *a** | *b** | *$DE_{cmc}$, deviation from the average tile colour* |
|---|---|---|---|---|
| 1 | 60.31 | 1.78 | 5.81 | 1.02 |
| 2 | 62.68 | 1.53 | 6.70 | 0.43 |
| 3 | 61.76 | 1.80 | 6.25 | 0.38 |
| 4 | 61.63 | 1.85 | 6.36 | 0.43 |
| 5 | 61.63 | 1.85 | 6.36 | 0.43 |
| 6 | 62.12 | 1.74 | 6.28 | 0.23 |
| 7 | 62.74 | 1.57 | 7.01 | 0.70 |
| 8 | 62.15 | 1.47 | 6.87 | 0.59 |
| 9 | 61.48 | 1.70 | 5.81 | 0.67 |
| 10 | 63.72 | 1.41 | 6.28 | 0.69 |
| 11 | 63.35 | 1.52 | 6.73 | 0.63 |
| 12 | 61.49 | 1.49 | 6.22 | 0.34 |
| 13 | 63.43 | 1.49 | 6.70 | 0.66 |
| 14 | 61.62 | 1.71 | 5.85 | 0.62 |
| 15 | 61.64 | 1.44 | 5.96 | 0.48 |
| 16 | 62.43 | 1.51 | 6.35 | 0.17 |
| 17 | 61.96 | 1.53 | 6.61 | 0.31 |
| 18 | 63.07 | 1.45 | 6.39 | 0.43 |
| Average | 62.18 | 1.60 | 6.36 | 0.51 |




**Figures 10 and 11. Variations in $DE_{cmc}$ for adjacent tiles against benchmark tolerances**

Figure 10 demonstrates that all the adjacent tiles have colour deviations greater than the average for the pristine tiles, and that 6 out of the 7 pairs have colour deviations beyond the maximum. A similar acceptable $DE_{cmc}$ can be derived from the averages of the tile measurements taken in-situ, Table 2. The average $DE_{cmc}$ is 0.86 units, the maximum is 1.73 units, shown in Fig. 11. Again all the colour deviations between the adjacent tiles are larger than the average deviation from the average colour, but only 2 of the 7 pairs are beyond the maximum.

**Table 2. In-situ tile colour data**

| *Tile Number* | *L** | *a** | *b** | *$DE_{cmc}$, deviation from the average tile colour* |
|---|---|---|---|---|
| 1 | 60.52 | 1.64 | 7.09 | 0.18 |
| 2 | 59.88 | 2.01 | 6.07 | 1.19 |
| 3 | 62.91 | 1.63 | 6.98 | 1.10 |
| 4 | 61.83 | 1.73 | 6.80 | 0.69 |
| 5 | 59.68 | 1.66 | 6.50 | 0.62 |
| 6 | 60.92 | 1.74 | 6.16 | 0.94 |
| 7 | 60.34 | 1.69 | 7.73 | 0.66 |
| 8 | 59.83 | 1.79 | 6.64 | 0.50 |
| 9 | 60.72 | 1.78 | 8.75 | 1.63 |
| 10 | 60.23 | 1.86 | 7.11 | 0.17 |
| 11 | 58.12 | 1.87 | 6.09 | 1.39 |
| 12 | 59.70 | 1.60 | 7.04 | 0.34 |
| 13 | 59.95 | 1.69 | 7.89 | 0.83 |
| 14 | 60.82 | 1.67 | 6.87 | 0.30 |
| 15 | 57.67 | 1.93 | 6.16 | 1.50 |
| 16 | 60.72 | 1.77 | 7.86 | 0.78 |
| 17 | 61.53 | 1.68 | 8.75 | 1.73 |
| Average | 60.32 | 1.75 | 7.09 | 0.86 |

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### 5.6 Laying of the tiles

There are common practices associated with laying tiles to prevent blocks of mismatched colour being laid, such as the blocking shown in Fig. 3. This practice, commonly accepted in the UK, involves opening two or three boxes of tiles and working from all these boxes, laying alternately from each box. BS 8000-11.1: 1989, covering workmanship on site, requires the colour and shade variation of tiles to be checked, and the thorough mixing of variegated tiles. This practice prevents the exaggeration of small differences in tile colour that would occur if each box were laid as a block.

This practice had not been followed during the construction of the tiled floor, although many of the large colour differences would still have been unacceptable even if the tiles had been randomized before laying.

### 5.7 Conclusions of investigation

At the conclusion of this study there was quantitative data to demonstrate that the floor contained tiles with significant colour differences between them, and that the tiles retained for replacement purposes would not match the tiles present in a large portion of the floor. These colour variations are likely to have been caused by inconsistencies in the manufacturing process.

The case study uses colour measurement and statistical analysis to quantify the colour differences and to establish retrospective tolerances on the colours. However, this detailed testing would never have been required if the acceptable colour range had been agreed, and measured by the client and the specifiers at the outset of construction. At the beginning of the project all that would be required would be the measurement of a few tiles representing the colour extremes acceptable for the floor.

Equally, if colour assessment had taken place for the boxes of tiles as they were opened, the tiles could have been selected more appropriately to produce a randomized effect across the floor. Tiles at the extremes of the colour range would not have been laid in adjacent positions.

## 6 CONCLUSIONS

The benefits of effective colour specification at the outset of a construction project are exemplified by the case study presented here. Action taken early in construction projects can assist in product purchase, execution and avoid the costly process of making good.

The significant body of data collected in this case study is able to benefit the construction community by providing examples of colour deviations that could be used as pass/fail criteria in the purchase of other coloured products. It is suggested that the acceptable $DE_{cmc}$ of 1.02 units is considered for materials with a coloured surface made up of dots between 2 mm and 5 mm, such as ceramic floor tiles and some stones and that any products exceeding this limit should not be adjacent..

As often the colour of the flooring, and other construction products, is of aesthetic importance to the client, there is no logical reason why the definition of colour should not form a regular part of the construction specification process.

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# An Integrated Approach To Durability Assessment Throughout The Construction Procurement

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**Summary: Durability assessment offers a mechanism by which to assess the durability risks associated with a building or structure, and to provide the desired function and design life by mitigation of these risks. It is intended as a practical tool for owners, facility managers, designers, contractors and any other party with an interest in achieving design life. Clients whose construction projects are reviewed benefit from a clear understanding of the structure they will eventually own. However, durability assessment can provide a competitive advantage to designers, specifiers and so on, by reassuring the client of the competence of the organisation. Equally, the designers, specifiers and so on, will benefit from the clarity of the client requirements and the discussion regarding the feasibility of these requirements.**

**Recent consultation with industry has demonstrated that the practice of durability assessment is increasing but that it still occurs in too few construction projects. However, there is a willingness to use this tool if cost-effectiveness can be proven. Durability assessment begins by the specification of durability in the client brief and is continuous though the life-cycle of the building or structure, until disposal. Although not required by the standard BS ISO 15686-3:2001, assessment of the conceptual designs is highly recommended to enable minimisation of the durability risks identified during the detailed design phase. This recommendation is exemplified by a recent case study.**

**Assessment of durability risks is an essential part of the process. The identification of such risks should pay attention to both the components elements and their interfaces. Risks are rated on the basis of the likelihood and consequences of failure, where the consequences include the costs of repair or replacement and the costs of disruption and loss of function.**

**Keywords: Durability, assessment, tools, risks**

## 1 INTRODUCTION

Durability assessment is essentially a check that a proposed building (e.g. residential accommodation, offices) or structure (e.g. bridges, roads) will be fit for its intended use and lifecycle. It is of growing importance to major construction clients. For example, HM Treasury's procurement guidance (1997) that applies to around £800 million of property procurement says decision-makers should ensure that "*risks have been properly identified, evaluated, allocated and managed effectively*" before investment is made.

Durability assessment is the consideration of design life at all stages of the construction process, from development of the client brief, through the design phases, construction phases and into the use and life care of the building or structure itself. It enhances communication throughout the construction process by providing a standardised method to consider and feedback durability information that:

- identifies risks which may impair durability and evaluates the response
- addresses unrealistic performance expectations on the basis of design information
- captures the on-going inspection and maintenance requirements to ensure the durability of the building or structure once it has been transferred from the construction team to the users.

It is a useful tool for owners, facility managers, designers, contractors and any other party with an interest in achieving design life. The principles of such assessments are set out in the recently published BS ISO 15686-1: 2000, BS ISO 15686-2:2001 and BS ISO 15686-3:2001.

## 2 THE BENEFITS OF DURABILITY ASSESSMENT

A recent industry consultation exercise conducted by BRE highlighted the potential benefits arising from durability assessment (Hooper and Rizzi 2001).

Benefits to the construction project

- Better understanding of project priorities and where valuable resources should be most effectively applied
- Clear guidance and allocation of responsibilities in the design and construction phases
- Increased clarity in the client's understanding of the scope of the design and construction process
- Reduced costs due to over-specification of the durability of materials or components
- Improved construction quality, particularly of the aspects critical to durability
- Improved risk management by quantifiable decision making
- Optimisation between capital expenditures and operational expenditures through whole life costing of construction solutions

Benefits to the clients and users

- Reduced costs associated with durability failure
- Planned maintenance scheduling and reduced disruption associated with repairs
- Reduction in risk and uncertainty and improvements in budgetary control

Benefits to the construction industry

- Creation of marketing opportunities though standardisation of approach and traceability
- Information to feedback to improving Building Regulations, materials specifications and design procedures
- Minimisation of waste and ineffective use of materials
- Achievement of more sustainable construction by minimisation of under and over-specification

Clients whose construction projects are reviewed benefit from a clear understanding of the structure they will eventually own. Ultimately, effective durability assessment relies upon the capability of the construction team (designers, specifiers, engineers and so on) to design and build durable buildings and structures. In this way the assessment can provide a competitive advantage to the construction team by reassuring the client of the competence of the organisation. Equally, the designers, specifiers, engineers and so on, will benefit from the clarity of the client requirements and the discussion regarding the feasibility of these requirements.

## 3 REQUIREMENTS OF DURABILITY ASSESSMENT TOOLS

The BRE industry consultation indicated (Hooper and Rizzi 2000) there is a clear understanding of the benefits of inclusion of durability specification in the project brief and durability assessment of the construction indicating the drivers for uptake of durability assessment methodologies are present. Importantly, the consultation indicated that the costs of durability assessment would be accepted if proven to be offset by reductions in running costs of the building or structure. This suggests that one barrier to implementation is the requirement to prove the cost benefits of assessment. However, despite the understanding of the benefits of durability assessment still too few assessments are actually conducted. This demonstrates the requirement to disseminate the benefits and methodologies for durability assessment and to develop practical tools to implement methodologies.

The consultation indicated that durability assessment tools should:

- be flexible to differing procurement processes such as Traditional Procurement and Public Private Partnerships,
- be effective as an in-house tool but understandable by clients, and useable by third parties if required,
- recognise the importance of economic life care requirements over the design life of the building or structure, since the end of service life is often related to uneconomical to operation or maintenance.

## 4 REQUIREMENTS OF CLIENT SPECIFICATIONS

The industry consultation conducted by BRE (Hooper and Rizzi 2000) indicated that over 50% of the respondents currently include, or seek to include, specific service life requirements in project briefs. A previous whole life costing consultation conducted in 1998 (Clift and Bourke 1999) indicated that only one third of briefs often or always contained this type of information. This indicates that there is increased use of durability requirements in specifications, for both Government and commercial clients. The study indicated that currently, where durability is specified in the project brief, designs are checked against this specification. Hence tools to assist this specification within the briefing process need to be developed to assist this definition by the client.

The extent of the information given to the construction design team is a decision to be made by the client, but can include:

- The function of the building or structure
- The design life
- The period of tenure and the required residual value at the end of this period
- The minimum design life
- Factors leading to the end of service life
- Optimising flexibility and minimising obsolescence
- The required level of inspection, maintenance and replacement or repair
- Design considerations such as the aesthetic requirements or sustainability credentials
- The budget considerations in place at the time of writing the brief

## 5 THE PRINCIPAL STAGES OF DURABILITY ASSESSMENT

Table 1 sets out the possible stages to be included in the durability assessment of the procurement process although not all of these assessments are required by BS 15686-3:2001.

**Table 1. Possible stages in durability assessment**

| *Assessment of* | *Reasons to assess* |
|---|---|
| The client brief | To ensure that the durability of the building or structure is specified, understood and agreed by both the client and the design team. This agreement ensures the client brief is a document that can be used as a benchmark against which to assess the later stages of the design and the construction process. |
| The conceptual design | To ensure that the specified durability requirements of the building or structure are fulfilled, by identifying the broad durability issues connected with the construction project before the detailed design process begins. This assessment is not required by BS 15686-3:2001 but is highly recommended as decisions that are made could have durability implications at a later stage. An overview of such an assessment is included. |
| The detailed design | To ensure that the specified durability requirements of the building or structure are fulfilled by the detailed design. BS ISO 15686 -1: 2000, BS ISO 15686 -2:2001 and BS ISO 156686-3:2001 contain substantial advice concerning predicting the service life of a detailed design. |
| The life care plan | To ensure the life care plan, developed from the detailed design, is in accordance with the requirements specified in the client brief and that this plan is understood and usable by the client. This is not required by BS ISO 15686-3:2001 but is again highly recommended. The assessment should include check that the plan is feasible and that it will provide the appropriate durability, but is not excessive. |
| The construction | To ensure that the building or structure has been reasonably planned so that durability critical areas can be and have been appropriately constructed (not required by BS ISO 15686-3:2001). |
| Life care | To ensure that the requirements of the life care plan are being fulfilled and that the durability of the building or structure has not been impaired by other works (not required by BS ISO 15686-3:2001). |
| Disposal/ deconstruction | To ensure the disposal of the building or structure does not have future implications for the durability of the site and that the disposal is as cost-effective as possible (not required by BS ISO 15686-3:2001). |

## 6 RELATIONSHIP TO CONSTRUCTION PROCESSES IN PLACE

### 6.1 Traditional procurement

Traditional procurement still forms the backbone of construction in the UK. The client develops a brief, the design team produces a detailed design for a buildable structure with specific materials and finally the construction team builds the design, handing the finished product over to the client or user. This process can be assisted by durability assessment at all stages. Durability assessment can be used as a tool to help the flow of information concerning durability throughout this procurement process. It is there to help:

- clients understand what durability information designers need to design the buildings clients require
- designers to ensure that their designs are buildable and functional with respect to durability
- contractors to plan the construction of aspects critical to durability
- users to schedule inspections, maintenance and repair to minimise disruption
- budget holders to plan for operational costs.

### 6.2 PPP, PFI and Prime Contracting

Public Private Partnerships -PPP (HM Treasury 1999), the Private Finance Initiative -PFI (HM Treasury 2000) and Prime Contracting (Holti et al 1999) are procurement routes for UK public sector construction. They aim to develop partnerships with the private sector and hence deliver improved value for money. Durability assessment can assist both the client and the bidder in demonstrating that the value for money objectives will be met. For example:

- Clients can use durability assessment as a tool to develop improved client briefs (the output specification) that address the fundamental durability requirements of the contract, without specifying inputs, which are the bidders responsibility. Assessment can also be used to identify, at the selection of bidders stage, which bids address the durability requirements of the building or structure most effectively
- Bidders can use durability assessment to review the client brief, to determine if the information required to proceed with the initial bidding phase is present and whether to bid for the contract. Additionally durability assessment can be used to demonstrate how the durability requirements in the client brief will be addressed in the construction process.

### 6.3 Whole life costing

Whole life costing has been identified as a mechanism to deliver improved value for money (Construction Clients' Forum 1999). It accounts for both the capital costs and the operating costs to develop the most cost-effective solutions. It is carried out for the client at various stages of the decision making process. The aim of refining the calculation of the whole life costs is to reduce the uncertainty inherent in the construction cost. Clearly identifying the areas of the construction which are fundamental to the durability of the building or structure will influence the choice of options under consideration and assist in refining the whole life cost calculations. Examples of the decision stages where both durability assessment and whole life costing overlap are:

- the assessment of feasibility of alternative construction solutions
- the choice of components and services during the conceptual and detailed design phases
- a suppliers decision to tender and tender appraisal by the client
- assessment of variations during the course of construction
- determination of operational costs incurred through life care activities.

### 6.4 Value management / engineering

Value management / engineering is a pro-active, creative, team approach to problem-solving in projects to provide the best value for money (BRE 2000). It considers alternative solutions that include options other than construction solutions but accounts for both capital and operating costs of construction solutions. In a similar manner to assisting the whole life costing process, durability assessment supports value management / engineering by providing a structured method to assess the durability consequences of the construction solutions, and hence their impact upon the desired building function.

### 6.5 Compliance to BS EN ISO 9001:2000

BS EN ISO 9001:2000 requires that the design and development process be planned, reviewed, verified, validated and any changes to the process controlled. This is achieved through the effective maintenance of records, checking of inputs and outputs, assignment of responsibilities and clear communication. Durability assessment can assist in compliance to this quality standard by providing a documented quality and review procedures that ensure the design product meets the clients requirements.

## 7 ASSESSMENT OF RISKS TO DURABILITY

BRE has developed the concepts of Durability Critical and Sensitive Areas, which underpin the risk assessment strategy of its durability assessment procedure. The concepts have been developed to enable rapid and cost-effective assessments to be conducted. The differentiation between Durability Critical and Sensitive Areas depends on the contribution of the area to the durability of the building or structure. The definitions of these areas are:

- Durability Critical Areas (DCA) are the parts or elements whose durability is critical to the performance of the whole building or structure. Failure of these parts or elements would lead to the immediate or eventual failure to provide the required minimum level of performance.
- Durability Sensitive Areas (DSA) are the parts or elements whose durability is not critical to the performance of the whole building or structure but are important to the ability to function effectively. Failure of these parts or elements would generally not lead to the immediate or eventual failure to provide the required minimum level of performance, although eventual failure is conceivable.
- Other areas not identified as either DCAs or DSAs are regarded as Unclassified Areas (UCA). Failure of these parts or elements would either be unlikely to occur or would not affect the ability of the structure to provide the required minimum level of performance.

Clearly, identifying the DCAs and the DSAs is a very important matter. However, the identification of the costs associated with the repair work, including the costs of disruption, in the event of a durability failure is also very important to this process and these should be taken into account. This implies that the definition of DCAs and DSAs and hence UCAs is a matter of risk assessment, that is likelihood of failure and consequences of failure, including the cost consequences. Examples of DCAs, DSAs and UCAs are given in Table 2.

### 7.1 Identification of durability risks

The BRE assessment procedure particularly segregates the *at risk* areas into two categories, *at risk* elements and *at risk* interfaces. Advice concerning the service life of components, that is building elements, can be assessed by referring to benchmarking documents such as the HAPM Component Life Manual (1992) or from the experience of the design team, including the materials suppliers. It is important to note that although there may be significant data available regarding the service lives of components, excessive regard to this elemental approach could mask the need for durable interfaces between components that will function for the required service life. The interface between two components not *at risk* can be a cause of failure. For example, the most durable reinforced concrete, well compacted to limit carbonation and chloride ingress, will fail quickly if the expansion joints between sections are poorly sealed and allow chloride penetration. In order to facilitate the determination of elements *at risk* the assessment procedure expressly sub-divides the building elements into the following categories:

- Mechanical and Electrical
- Frame and Envelope
- Groundworks
- Internal Finishes

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**Table 2. Examples of DCAs, DSAs and UCAs**

| *Category* | *Example* |
|---|---|
| DCAs | <u>Membrane on a flat roof</u> |
| | failure causes water ingress to school classroom making the room unusable. |
| | <u>Flooring in entrance lobby</u> |
| | excessive wear causing reduction in slip resistance and making the floor hazardous. |
| | <u>Expansion joints between sections in a bridge deck</u> |
| | failure of sealants leads to reinforcement corrosion, ultimately risking structural integrity and excessive repair costs and disruption. |
| DSAs | <u>Felt on a flat roof</u> |
| | failure causes water leakage into open bus shelter making the shelter damp and causing damage to internal finishes. |
| | <u>Service pipes under entrance to building</u> |
| | failure will mean mosaic floor must be lifted at great expense and significant disruption. |
| | <u>Thermal expansion mismatch between blockwork walls and tiling in hot kitchen facility</u> |
| | causes some tiles to become loose, causing inconvenience and a potential hazard to users. |
| UCAs | <u>Sulfate attack of above ground concrete structures</u> |
| | is unlikely to occur. |
| | <u>Improper fixing of door handles</u> |
| | is unlikely to occur and is easily rectified without disruption. |

These sub-divisions form the same supply chain *Clusters* as in Prime Contracting Procurement (Holti et al 1999). This correlation is specifically intended to maximise the benefits of Prime Contracting by using the specialist knowledge available by integrating the supply chain.

The various elements within the design are assessed against the specifications of the client brief. That is, will they match the durability and life care requirements specified by the client? Those that fail to match the requirements are *at risk*, for example, window frames that require re-painting every 4 years are *at risk* if the life care requirements only allow for preventative maintenance every 5 years.

Following this elemental analysis, the same procedure is followed with each of the group interfaces being analysed for risks in a similar manner, including analysis of interfaces between elements in the same sub-division. When specialists are identifying the DCAs and DSAs, it is pertinent to allow each to assess the design individually, from their own specialist knowledge, before comparing notes and agreeing a comprehensive list of *at risk* interfaces. Table 3 shows how this process works.

**Table 3. Elemental and interface assessments required**

| *Elements "at risk"* | | *Interfaces "at risk"* | | | |
|---|---|---|---|---|---|
| | | *M&E* | *GW* | *F&E* | *IF* |
| Mechanical and Electrical (M&E) | X | X | | | |
| Groundworks (GW) | X | X | X | | |
| Frame and Envelope (F&E) | X | X | X | X | |
| Internal Finishes (IF) | X | X | X | X | X |

### 7.2 Assessing the consequences of durability failure

Once the *at risk* elements have been identified consequences of failure are assessed. The consequences should specifically relate to the ability of the building or structure to perform in accordance with the requirements of the client brief. At minimum, such assessment should include to consequences in terms of:

- Minimum level of performance required
- Impact upon structural integrity
- Financial consequences of repair or maintenance

- Disruption to function caused by repair or maintenance
- Reasons for initial design preference
- Forecast obsolescence or changes in circumstances

**Table 4. Assessing of consequences of failure against the client brief**

*Specified requirements in the client brief:*

- Required service life of whole building is 35 years.
- Routine maintenance operations to be conducted every 5 years.
- Major replacement operations after 10 years, to a maximum cost of £ 70, 000 for remaining life of building.

*Assessment of consequences:*

| | |
|---|---|
| Minimum level of performance required | • Require maintenance every four years to match design life<br>• 5 yearly maintenance specified in client brief<br>• With 4 yearly maintenance service life will be 25 years<br>• Without maintenance service life will be 10 years |
| Impact upon structural integrity | • No impact |
| Financial burden of repair and/or maintenance | • Cost of four yearly maintenance = £1000 + 3%<br>• Cost of five yearly maintenance = £1100 + 5%<br>• Cost of replacement after 25 years = £10,000<br>• Total cost of replacement every 10 years = £20,000 |
| Disruption to functionality caused by repair or maintenance | • Limited disruption from four/five yearly maintenance<br>• Large disruption to function through replacement |
| Reasons for initial design preference | • Design restrictions imposed by client:<br>• Wooden window frames from renewable source<br>• Aesthetics of design with local area |
| Forecast obsolescence or changes in circumstances | • Potential extension to building, to accommodate more staff after 10 years, from south elevation |
| Conclusion | Wooden window frames are a DSA |

Again, using the example of the *at risk* window frames, the client brief is used as the standard against which to measure the consequences, demonstrated in Table 4. This analysis of consequence highlights these window frames as a Durability Sensitive Area. That is, the level of life care requirements specified in the client brief prohibits the required maintenance, and the life care specified is likely to lead to failure to provide the required service life after 25 years. However, the choice of windows is limited by the design restrictions imposed by the client. In this situation the windows will not impact on the economic function of the building and the replacement costs, and associated disruption, are acceptable if planned for. Hence, there is recognition of the mismatch in terms of durability but also that it is not critical to the building performance.

Having highlighted the window frames as a DSA this is fed back into the design process. In this way the options to rectify the issue can be addressed, for example:

- The client may agree to four-yearly maintenance of the windows to achieve desired service life, stimulating a change in the client brief.
- The costs of replacement after 25 years will be accounted for in the through life costs of the design.
- The inspection and maintenance plan will eliminate the requirement for painting of the windows on the south elevation.

## 8 CASE STUDY - DURABILITY ASSESSMENT OF THE CONCEPTUAL DESIGN

Recently, the BRE used durability assessment to identify and assess the risks related to function and buildability of a design of a medical practice. The study highlighted areas of concern within the conceptual design that should be addressed through the detailed design. The assessment process was able to form a discursive link between the client and architects. For maximum impact durability assessment was complemented by energy efficiency assessment and whole life cost analysis.

The client was involved in the refitting a medical practice through a lease arrangement and although the clients would not be responsible for the capital costs they would be responsible for maintenance, refurbishment and running costs during the period of the lease. There were obligations imposed on the design by UK Health Authority guidelines with regard to long term performance, and the clients had expressed a preference for reduced environmental impacts if cost-effective.

The durability assessment located several areas that will be vulnerable to detailing and workmanship issues. The areas critical or sensitive to durability included:

<u>Rooflight upstands</u> – issues on both weatherproofing and security.

<u>Downpipes through flat roofs / floors</u> – requiring initial weatherproofing and regular maintenance to remove leaves etc.

<u>Edge details around flat roofs</u> – parapets should be avoided, and gutter sizing should be generous.

<u>Areas below kitchens / bathrooms</u> (especially floor-drained showers), including sealing details around sinks, basins and baths, and drainage details from shower trays and washing machine outlets (which should be accessible for clearing)

<u>Pipework in screeds</u> – contact with wet concrete can cause problems to copper or plastic pipework.

<u>Sealing of gaps between walls and floors</u> – particularly important in the consulting rooms for noise reduction and confidentiality

The recommendations to minimise durability risks included:

- Review use of roof lights and consider use of light tubes cast into roof slab.
- Provide inverted flat roof construction to flat roof / terrace area without parapet.
- Consider linoleum floor finishes on the basis of cost / hygiene / reduced environmental impact with sealed upstands and skirtings. Consider wall hung cupboards, sanitary ware and so on that is faster and easier to clean around.
- Where pipes are laid in floors an accessible ducting arrangement should be considered.
- Plan for maintenance of joinery and ironmongery, a system which allows minor repairs is to be preferred to one where major replacements are foreseeable, but these are critical to access and usability of the building, so should be a priority in terms of considering performance. Similarly, factory finishing can provide a longer interval to first maintenance. Consistent use of specifications reduces unfamiliar operation and the need to hold different spares.

## 9 CONCLUSIONS

Durability assessment is a powerful but underused tool for the construction industry. It not only has a role to play in establishing durable and functional buildings and structures it can assist in integrating the supply chain, procuring construction projects and bidding to win these projects, providing value for money and complying with quality standards related to product design.

Consultation with the UK construction industry has demonstrated that although consideration of durability in the procurement process is increasing it is only sought in 50% of projects. However, this is an increase over previous measures and is indicative of the growing recognition of the importance of durability assessment and other initiatives such as whole life cost analysis, value management / engineering and partnering in construction procurement.

Although not required by BS ISO 15686-3:2001, the assessment of the conceptual design is highly recommended to minimise the durability risks as an early stage and focus the detailed design stage. Structured and documented risk assessment is the foundation for durability assessment. This risk assessment should include consideration of the likelihood and consequences of failure, where the consequences include the costs of replacement or repair and the costs of disruption.

## 10 ACKNOWLEDGEMENTS

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# IMPACT STRENGTH OF MODIFIED WOOD SPECIES

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**Summary: At present commercial methods are available to improve the durability of wood by hydro-thermo-mechanical action without any preservative chemical component. The methods aim at modifying part of the cell fibre structure that provide the food for wood deteriorating organisms. This opens market perspectives for timber where environmental demands can no longer be met by traditional chemically treated wood. Besides the durability also swelling and shrinkage and the UV-sensitivity is considerably reduced. Although this modified wood might be considered as an alternative structural material, it should be taken into account that some of the mechanical properties are changed. Over 4000 km of galvanised steel guard-rail is located along the major traffic roads in the Netherlands. An innovative design for an alternative timber guard-rail was made and a pilot project set up. Although the durability requirements could well be met by the modified wood the ability to withstand impact loads was questioned. For this reason impact bending tests were carried out covering seven wood species of various grade including three modified wood species. Some wood species didn't show any significant bending strength reduction while others like the modified wood species showed a 50% reduction.**

**Keywords: Timber, impact, bending strength, durability, modified wood.**

## 1 INTRODUCTION

In the Netherlands there is a governmental incentive to stimulate the use of renewable raw materials in building. For this reason initiatives were taken to set up pilot projects to show the viability of timber for structures that normally are designed in steel. One of the projects is the design of a timber guard-rail for heavy traffic roads as an alternative for the traditional steel guard-rail. Reason being the water pollution caused by the fact that the zinc coating of the steel boards drops off after a number of years in service. Although the amount of zinc might seem low per meter, multiplied with 4000 km of guard-rail these quantities become considerable. Zinc coating that lasts longer costs much more and therefore timber might well be considered too. After a thorough design process the guard-rail shown in Fig. 1 was finally chosen. The curved parts should be able to take impact loads of small vehicles while the sturdy poles in the middle take the high load from busses and lorries.

Durability aspects particularly if designed in timber are very important. Maintenance schemas are set to 15 or 20 years indicating that large-scale repair was unwanted at shorter intervals. Considering these demands the choice of wood species became important besides the availability of the species. Now here is where the new heat modified wood species came to mind again. However, unlike most civil engineering structures the relevant type of loading for guard-rails is impact. Old test data shows that timber is well able to withstand impact-bending loads. Would the heat-treated products be able to resist impact load as good as the untreated wood? To answer this question an investigation was set up testing seven wood species in impact bending of which three were of the new heat modified type.

## 2 WOOD MODIFICATION

Wood has a polymeric cell wall structure, which contains cellulose, hemicellulose and lignin as the main constituencies. The water absorption and therefore the swelling and shrinkage of wood is due to the forming of hydrogen bounds of water molecules between the hydroxyl (OH-)groups of these constituencies as water molecules needs more space. Thermal modification of wood is a treatment where the cell wall polymers are altered by heat. This can lead to cross linking of the OH-groups and therefore to a reduction of the potential positions for water molecules. In other words the hygroscopicity is reduced. Another result is that molecule chains are cut in smaller pieces. A well-known advantage is the reduced sensitivity to bio degradation. However, a draw back is the loss of strength and increased brittleness. However, the last decade new types of thermal treatment were investigated which claim the have minimised the negative effects to a considerable extend. Examples of new thermal treatment methods are known as Perdure (France), Stellac (Finland) and the Plato (Dutch) process. Wood treated according to the Plato process has a better resistance against brown rot, fungi, white and soft rot, Homan et al. (2000).

The average loss in bending strength vary from 5% to 18% which is less than other thermal treatments. The potential market is cladding, garden fences, furniture, poles, sheds and retaining walls for canals.

## 3 THE IMPACT STRENGTH OF TIMBER

During the Second World War, when wood was used to a considerable extent for structural members for training aircraft's and gliders, it became evident that additional test data were necessary to improve the understanding of the behaviour of wood under impact load. Therefore, a comprehensive test program was initiated at the Forest Products Laboratory, Madison in the US, to study the effect of rate of loading on the bending and compression parallel to the grain, Liska (1955). The planks were small in size and free of defects and as straight grained as possible. Loading times ranged from 0.3 to 150 seconds. The bending tests were performed in a hydraulic testing machine with a constant head movement. The average of the controls (references) mentioned in Fig. 2 refer to the standard bending tests of Maple. The data for all wood species tested follow the same tendency. At the highest loading rate the strength is about 20 to 30% higher than the standardised bending strength.

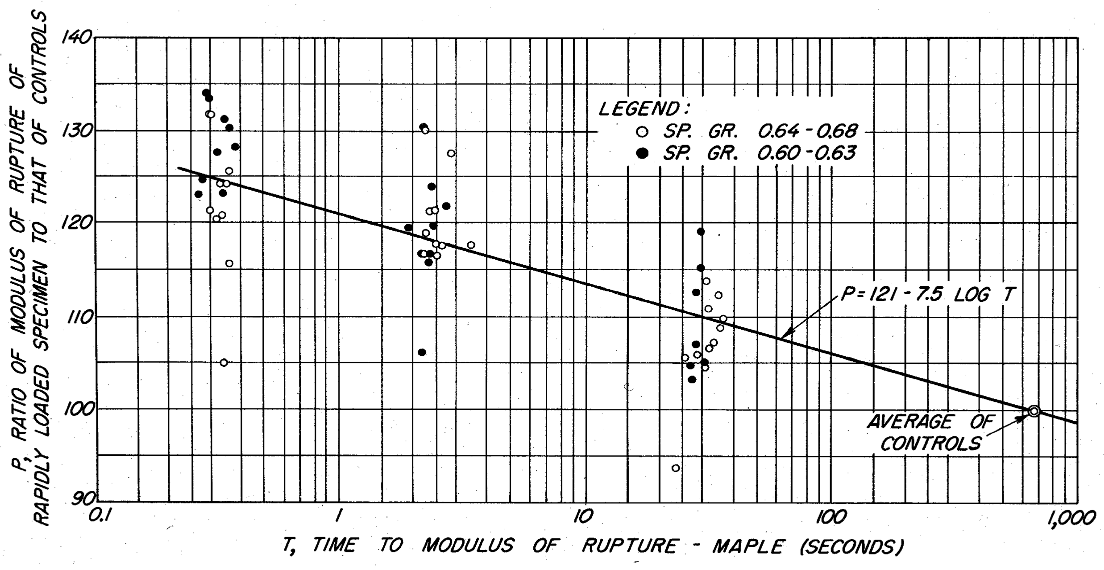
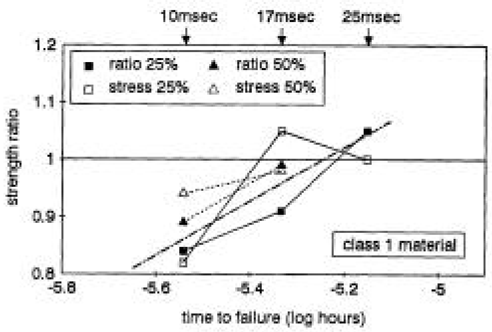



**Figure 2: Ratio of static and impact bending strength versus loading rate for Maple, Liska (1955).**

More recently Madsen et. al. (1986) and Jansson (1992) studied the impact bending strength on simply supported timber beams by a weight dropped from various heights. The drop height of the 345 kg weight varied from 50, 150 and 300 mm height resulting in a maximum impact velocity of 2,3 m/s. The average times to failure were 25, 17 and 10 milliseconds. Essential in these tests by Jansson (1992) is that the impact force was measured directly by means of a load cell between the drop weight and the test specimen. In the analyses of the results the importance of separating the applied load in a part, which introduce bending stresses and a second part which sets the beam into motion is demonstrated. It should be pointed out that in former timber tests the inertia forces were always disregarded, as they were assumed negligible. Analytical procedures earlier to estimate the inertia effect were explored but rejected. Jansson (1992) turned to a Modal Analysis approach to tackle this problem. Some of his experimental results are given in Fig. 3. The impact bending strength decreases with decreasing time to failure. The deviation from the test results mentioned earlier is considerable. No strength increase of 20 to 30% but a strength decrease of 15% for the shortest loading time than 10 milliseconds was observed.

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## 4 IMPACT TEST

The aim of the research was to compare the static and impact bending strength in flat wise bending of a number of wood species.

### 4.1 Rate of impact loading

In the European guard-rail test standard EN 1317 a number of performances levels are specified. The performance level H2 of the Dutch guard-rail was arbitrarily set by the authorities. The two tests prescribed for H2 level are full-scale crash tests with a heavy 18-ton bus and a care. The bus shouldn't break through the structure while for the care test the acceleration of the passengers is limited. As the bus crash tests is regarded as the governing test case for strength the loading speed in the impact tests was deduced from this test. The bus entrance speed is 70 km/h (19.4 m/s). It will hit the guard rail at an angle of $20^0$ which leads to a lateral speed of 19.4 sin $20^0$= 6.7 m/s. For this reason a load speed of 7.0 m/s was chosen in the impact test.

### 4.2 The test apparatus and instrumentation

In principle the test apparatus consists of a weight that is dropped from a height and that hits the simply supported beam, span 1400 mm, at mid span. The total weight of the drop piece was 199.0 kg. The weight was instrumented with an accelerometer. The de-acceleration of the drop weight, thought to be a key to determine the excitation force didn't work. As shock waves in the weight itself overruled the de-accelerating signal completely. A transducer (LVDT) attached to the bottom side of the timber specimen directly underneath the impact location, Fig. 4, took the beam deflection. This LVDT was hidden in a stronghold below the specimen to prevent any damage of the device after failure of the specimen. A high-speed video camera (9000 frames per second or one frame every 0.111 ms) enabled monitoring the behaviour during the test. Failure was defined as t he visual appearance of the first crack.

**Figure 4: Transducer attached to specimen**

### 4.3 The wood species

The number of specimens per wood species was limited to twenty per wood species. Seven wood species were chosen, Angelim Vermelho (tropical wood), Douglas Fir, Ash, Larch and three so-called heat treated or heat modified wood species. With the PLATO process Douglas Fir was modified (PLATO is a Dutch patented process) while Spruce (Picea Abies) and Pine (Pinus Silvestries) were modified using the Finnish STELLAC process. Clear free were Angelim Vermelho, Douglas Fir and Ash while the others were of a commercial grade including knots and other deficiencies. The PLATO wood was of the lowest grade involving the biggest knots. In the evaluation of the test results these wood species are referred to as Mod.I to III species. All specimens were conditioned at 80% RH and $20^0$ C. The batches for standard and impact bending were matched on the basis of MOE.

## 5 STATIC BENDING TESTS

The standardised static bending tests were carried out corresponding to EN 408. Three-point bending was chosen as it corresponds with the impact test. The specimens of about 1600 mm length were symmetrically positioned on the supports of 1400 mm span and loaded until failure. The load deflection curve was recorded.

### 5.1 Impact tests

In the impact tests an unexpected phenomenon was recorded by the high speed camera, which appeared to be of importance for the simulations and interpretation of the test results. The pictures clearly showed that after an initial contact and impulse transfer the beam accelerated more rapidly than the weight and lost contact. The impulse transfer was apparently so high that the beam speeded up more than the drop-weight fell. After a short time the drop-weight made contact again and transferred a second impulse. The second separation between both was much smaller and shorter than the first time. Finally, the drop-weight

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established a permanent contact and worked its way down until failure of the beam occurred. In Fig. 5 two curves are drawn. The bottom one resembles the output signal of the accelerometer attached to the weight and the second one is the deflection both versus time. On the left axis the beam deflection in given in millimetres. The bouncing effect can be observed in this deflection curve, as it isn't a straight curve. Certainly the first bump is clearly visible. The deflection data was used for computer input to simulate the test and find answers to what the bending stress was at time of failure. Janssen (1992) showed that the previous applied analytical methods to determine the bending stresses from the experimental data were questionable. In pursuit of a more reliable method a computer simulation programmes was adopted. The tuning of the simulation programme to fit the recorded test data is obviously very important. In this programme the inertia influences are taken into account in a more accurate manner than in the analytical methods.

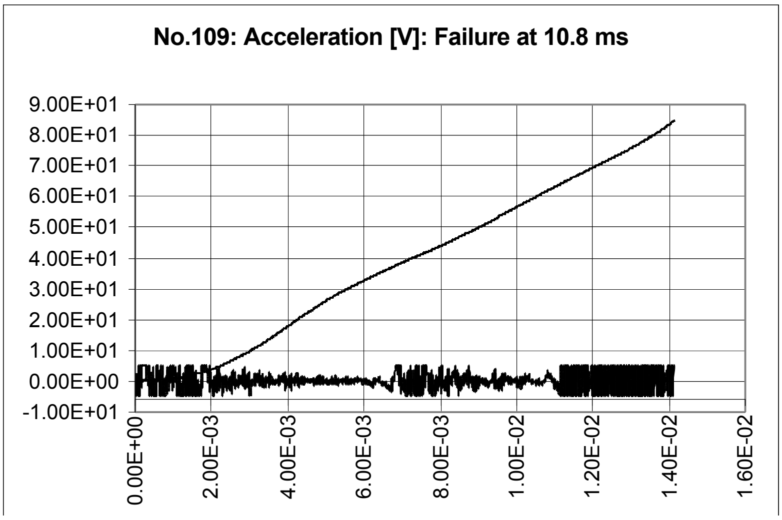


**Figure 5: Deflection [mm] versus time [s] at the bottom the accelerometer signal**

The FEM simulation model was based on Timoshenko elements. A. Kok (1997) developed this particular simulation model suited to load the beam by impulses and to attach lumped masses at any given time. The model accounts for all inertia effects. A single impulse at the beginning of the simulation represents the initial contact and after a given time the drop-weight, characterised as a lumped mass of a certain quantity and velocity, can be attached to mid span elements. Damping can be introduced to diminish the effect of higher order vibrations. For every specimen the relevant material properties, MOE, density and dimensions were given in the input file. The simulation generated deflection, rotation, bending moments and shear forces. It allowed plotting the simulated time deflection curve, which can be laid over the time deflection graph recorded as shown in Fig. 6.

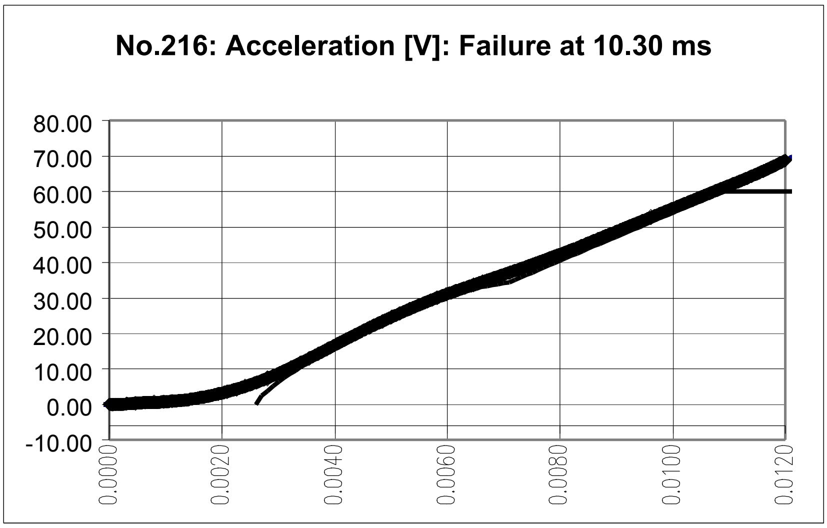


**Figure 6: Test and simulation result of deflection [mm] versus time [s]**

As shown in the first 2 ms the initial impulse is transferred to the beam during this time. It will be clear that the mathematical beam in the simulation exposed to an impulse reacts immediately. To have agreement the initial contact in the simulation is delayed. The delay time and the initial impulse are chosen such that both experimental and simulation time deflection curve are in good agreement. What happens in the first 2 ms is a point of interest. During this time span the beam accelerates to keep up with the speed of the drop-weight as the drop-weight is dominant in mass. The drop-weight will hardly slow down to allow the beam to speed up. During 0.002 ms the drop-weight will travel about 0.002x7000 mm/s = 14 mm. As Fig. 6 shows the beam deflects about 5 mm in that time span still leaving a difference of 9 mm to be explained. The top surface of the beam must have been penetrated by the drop-weight. However, inspection of the contact surface did not reveal a lasting mark of any reasonable depth (< 1 mm). No other effect could be thought of to explain this phenomenon. Perhaps at these loading rates the behaviour of timber is different of what happens at low loading regimes. The video camera could not give conclusive evidence of such penetration, as it was not focussed at this spot. It was noted that the contact between the specimen and the drop-weight was longer than expected and that it corresponded well with the LVDT readings. As impact test of concrete showed a much more prolonged elastic behaviour under this type of loads, timber might behave similar.

Having set the simulation input variables such that the deflection versus time corresponds well with the test data the model generates the corresponding bending stresses at any location along the beam. The overall majority of beams failed at mid span. For that reason the mid-span bending moment was taken as the governing value for the derivation of the bending strength. Only in some cases knots and slope of grain caused failure initiation a distance from mid span. In Fig. 7 a plot is given of a typical mid span bending moment versus time plot. The maximum bending moment is given in the top right corner in Newtonmeter. The bending stress is derived in the traditional way assuming Hook’s law applies for these conditions.

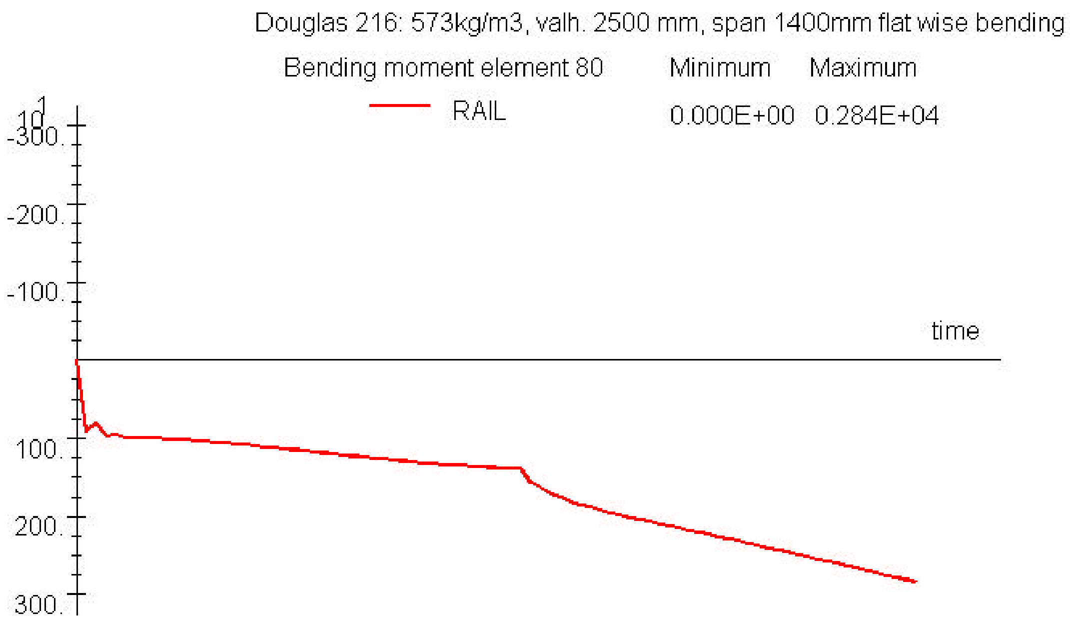


**Figure 7: Simulated bending moment [Nm] versus time [s] at mid span**

## 6 EVALUATION OF THE RESULTS

The results are graphically represented in Fig. 8. The strength ratio impact bending / standard bending is given in the lower part of Fig. 8. The wood species associated with the column numbers are given in Table 1. Notices the ratio in Column (3) for Ash, which is higher than 1, while for all other wood species it is considerable smaller than 1. Table 1 contains the results of the simulation per wood species and complimentary the data of the standard bending tests. To drawn more reliable conclusions only based on differences in mean bending strength the statistical t-test was applied. The analysis concludes that with 95% certainty the mean bending strength of Angelim Vemelho, Larch, Mod. I., Mod. II and Mod III wood species in impact bending is indeed significantly different from the standard bending strength. No significant difference was found for Douglas Fir and Ash.

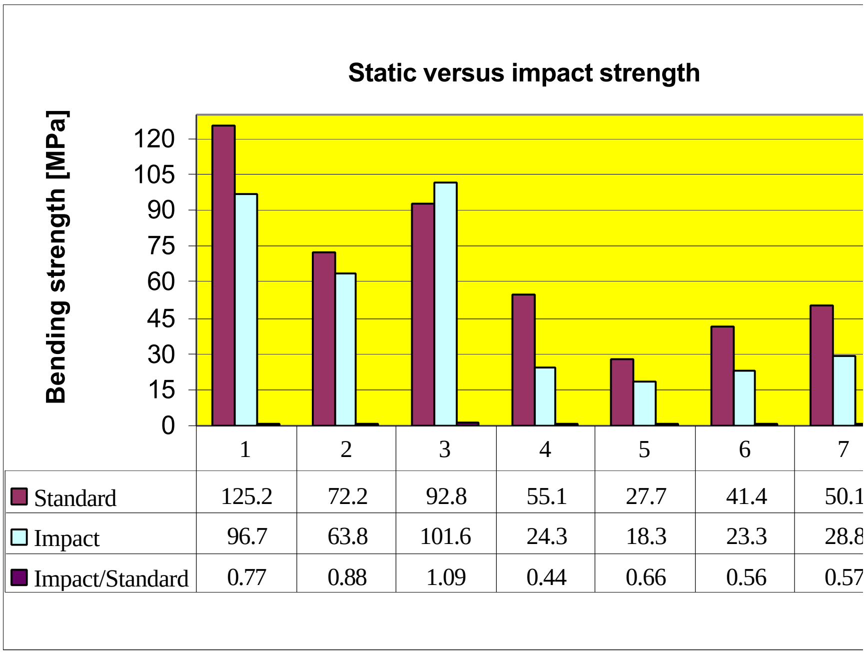


**Figure 8: Overview of the impact and static strength results**

**Table 1: Results of standard and impact strength tests**

| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
|---|---|---|---|---|---|---|
| Wood species | number of tests | Standard bending | c.v. | number of tests | Impact Bending | c.v. |
| | n | [MPa] | [%] | n | [MPa] | [%] |
| Angelim Vermelho (1) | 11 | 125.2 | 13 | 10 | 96.7 | 38 |
| Douglas Fir (2) | 10 | 72.2 | 14 | 10 | 63.8 | 39 |
| Ash (3) | 4 | 92.8 | 9 | 7 | 101.6 | 12 |
| Larch (4) | 11 | 55.1 | 26 | 10 | 24.3 | 29 |
| Mod.I (5) | 12 | 27.7 | 23 | 11 | 18.3 | 37 |
| Mod II (6) | 13 | 41.4 | 24 | 22 | 23.3 | 20 |
| Mod III (7) | 17 | 50.1 | 24 | 14 | 28.8 | 27 |

The difference between standard and impact strength has some relation with grade quality as the wood species of commercial grade (containing knots) reduced more in strength than others. Apart from Ash the other wood species free of any knots such as Angelin Vermehlo and Douglas Fir, dropped in bending strength only 23 and 12% respectively. The other wood species with knots such as Larch and the heat-treated modified wood species bending strength dropped 40 to 60 % in strength. The influence of knots turns out to be of significant importance. This agrees with the conclusion by Jansson (1992) that commercial timber behaves worse than clear and free wood specimens.

## 7 SUMMARY AND CONCLUSIONS

Standard and impact bending tests have been performed on matched timber specimens in flat wise three point bending. The span was 1400 mm while the specimens had a cross-section of about 40x130 mm. Seven wood species were investigated Angelim Vermelho, Douglas Fir, Ash, Larch and three heat modified wood species of Pine, Spruce and Douglas. The number of specimens per wood species in the static and impact test was 10 with some exceptions.

The standard bending tests were performed according to EN 408. For the impact tests a tailor made apparatus was built in absents of any standardised apparatus. The load application consisted of a 199 kg drop-weight which fell from 2.5m height and reached a speed at impact of 7 m/s. During impact the deflection was recorded at mid span. Failure was defined as the occurrence of the first crack. A high-speed camera (9000 frames a second) was used to record the time to failure. In order to determine the bending strength a simulation programme was used. The dynamic programme was tuned in such a way as to simulate the most important phenomenon observed during the impact test up to failure.

Reviewing the strength values derived it was concluded that the mean impact bending strength is in most cases lower than the static bending strength. It was concluded that no significant bending strength difference could be demonstrated for Ash and Douglas Fir. However, all other wood species especially those of commercial grade (containing knots) showed a considerable reduction in strength from 40% up to 60%. Despite the low number of tests the conclusions are in line with the results of Jansson (1992). Furthermore, it was concluded that all modified wood species failed in a very brittle way with short fibres while the drop of bending strength was considerable.

Although from a durability point of view modified wood species have good prospects and may open new markets, their poor behaviour to resist impact loads should be considered when applied in structural engineering. As a result of these tests the use of these modified wood species was no longer considered as a viable option for the timber guard-rail design. It was finally decided to apply tropical hardwood (Angelim Vermehlo) for its high (but reduced) strength in order to limit the cross-sectional dimensions of the timber members and for durability considerations.

## 8 ACKNOWLEDGEMENT

Thanks to the laboratory staff of Steel & Timber structures. Especially, Hylke Katsma and P. Stolle for their assistance in testing and active participation in solving all practical problems. I also want to express my thanks to Dr. A. Kok for his stimulating help applying his simulating programme. Furthermore, I would like to acknowledge the ministry of Traffic and Waterways DWW and Centrum Hout for their financial support in financing this research project and also to WIT Press who agreed in publishing part of Leijten (2001).

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# Potential Curvature Method - A New Approach for Corrosion Assessment in Concrete Structures

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**Summary: The half-cell potential survey method is a widely used technique for corrosion risk evaluation in reinforced concrete structures. ASTM C876 has appended to it guidelines for corrosion risk assessment based on three ranges of potential values. However, it has been found that the actual potential ranges that characterize the different levels of corrosion risk can vary significantly for concrete structures subjected to different environmental exposures. The ASTM C876 criteria can be misleading when used for corrosion risk assessment in different concrete structures. This paper describes a new analytical approach, namely the *Potential Curvature Method*, developed for corrosion risk evaluation based on half-cell potential surveys. The curvature map can effectively define those anodic sites that demonstrate both higher potential gradients and more negative potential values. This method has been successfully used in ACCI site investigations of concrete structures. The *Potential Curvature Method* has been found to be an effective tool for corrosion risk evaluation in concrete structures regardless of whether the structures are in the marine environments or mainly affected by carbonation**

**Keywords: Corrosion risk, Concrete, Half-cell Potential, Gradient, Curvature**

## 1 INTRODUCTION

The corrosion of steel reinforcement in concrete structures is a worldwide problem and has been estimated to cause a multi-billion dollar economic loss each year. Over the past four decades, it has attracted more and more attention from scientists and engineers concerned with the diagnosis, control and prevention of corrosion. However, generally accepted theories and mechanisms have yet to be established to answer some fundamental questions such as: how half-cell potential values can be correlated with the active corrosion sites; how steel passivation is influenced by the concentration and mobility of chloride ions, and how cracking in concrete and the crack width affect the long term durability of concrete structures.

The corrosion system of steel in concrete is more complicated than most other aqueous corrosion systems. The electrolyte in the steel-concrete system is normally the concrete pore water which can vary significantly not only due to the cement chemistry and the porosity of concrete but also due to local features (eg. concrete cover depth and presence of cracking) and local environmental factors (eg. humidity and chloride content). Further understanding of this complicated system is necessary before sound theories for the corrosion of steel in concrete and its prevention can be established. Consequently, for the time being, engineering practices based on empirical results will continue to play an important role in this field.

The testing of half-cell potentials has been a common technique for corrosion risk evaluation in concrete structures. Guidelines are proposed in the standard ASTM C876 for the interpretation of potentials in relation to the risk of corrosion activity. However, questions have been raised since the early 1980's (Brown et al, 1983) regarding the suitability of these criteria for concrete structures in different exposure environments. In addition, there has been some debate regarding the validity of using potential values for the assessment of corrosion risks at all.

This paper describes the development and application of a new approach, the *Potential Curvature Method*, for corrosion risk evaluation in concrete structures based on half-cell potential surveys. This new method is based on the gradient and curvature characteristics of the potential field rather than the potential values themselves, to identify the locations with higher risks of active corrosion in concrete structures.

## 2 HALF-CELL POTENTIAL TESTING - ASSESSMENT CRITERIA AND LIMITATIONS

The half cell potential technique became popular for corrosion evaluation in concrete structures after the investigation of 473 highway bridge deck slabs by the Federal Highway Administration (FHWA) of the United States in the early 1970's (Stratfull, 1973 & Van Daveer, 1975). On the basis of this work, guidelines were empirically developed for the interpretation of half-cell potential values in relation to the probability of active steel corrosion in concrete bridge decks. These guidelines are appended to ASTM C876 "Standard Test Method for Half Cell Potentials of Uncoated Reinforcing Steel in Concrete" for the evaluation of corrosion risk. They define three ranges of half-cell potential values, which are referenced to the copper/ copper sulfate electrode (CSE).

- those more negative than -350mV > 90% probability of active corrosion
- those between -200 mV and -350 mV 50% probability of active corrosion
- those more positive than -200mV < 10% probability of active corrosion

The majority of the bridge decks investigated by the FHWA had been subjected to de-icing salts. Researchers have subsequently found that the risk related potential ranges shift significantly for concrete structures under different environmental conditions such as in marine environments or where concrete is mainly affected by carbonation (Borgard, 1991; Elsener et al, 1990). Brown et al (1983) discussed three major effects — junction potentials, resistive concrete and polarisation fields — that could significantly affect the half-cell potentials measured on concrete structures. In marine environments, very negative potential readings can be found in saturated situations. However, the corrosion rates in these cases are often so low as to be negligible due to the lack of oxygen for the cathodic reaction. On the other hand, active steel corrosion can occur in carbonated concrete even when much less negative potentials are measured.

The half-cell potentials measured on concrete structures are in fact "*mixed potentials*" which are affected by both the anodic and cathodic areas of the reinforcing steel and the macrocell effect between them. In recent ACCI research work (Chang et al, 2000), a difference of more than 200 mV was observed between the rest potentials of two steel rebars (one in sound concrete and the other intersected by a crack) when the concrete specimen was immersed in a 3% NaCl solution. However, when the two rebars were electrically connected, a macrocell formed, with the rebar intersecting the crack being the anode; the mixed potential of the connected rebars also shifted to a value between the two potentials measured before their connection. In further ACCI research work on the macrocell effect (Chang et al, 2001), the steel rebars in uncracked concrete specimens immersed in seawater were measured with potentials of about –500 mV (CSE), while the rebars in cracked concrete specimens had potentials down to about –750 mV (CSE). It was also found that the cathodic steel surface in uncracked concrete but adjacent to the anodic surface in the crack zone gradually developed into a more anodic state due to the macrocell effect.

The criteria of ASTM C 876 may well give valuable indications when they are applied to very similar configurations of reinforcing bar and cover subjected to similar environmental conditions as those FHWA bridge decks. However, they have been found to give misleading assessments when applied to a range of structures which were very different to those FHWA bridge decks.

## 3 POTENTIAL CURVATURE METHOD - A NEW APPROACH

Because the potential value and the risk of active corrosion in concrete structures can vary significantly under different environments, the analysis of potential results based on local potential gradients rather than potential values has been recommended as a better way to identify active corrosion sites (Elsener and Bohni, 1990). Broomfield (1997) proposed that active locations in carbonated concrete might be indicated by a potential difference of 150 mV or more over a space of one metre or less.

A half-cell potential contour map can provide a graphical view of the potential gradient distribution within an area. Locations with denser potential isolines indicate higher potential gradients. However, these locations are not all related to higher corrosion risk. High potential gradients occur at two typical locations, with either more anodic or more cathodic characteristics relative to their surroundings. Higher risks of active corrosion are however only associated with those anodic sites, which are characterised by more negative potential values as well as higher potential gradients. Consequently, care must be taken when making corrosion risk assessments simply based on potential gradients viewed graphically from potential contour maps.

A new approach has been developed at the Australian Centre for Construction Innovation (ACCI) for identifying active corrosion sites based on half-cell potential results. The theoretical basis of the new approach is that higher positive curvature values pick up the more anodic locations characterised with both higher potential gradients and more negative potential values. The approach is therefore named the *Potential Curvature Method*.

To carry out on-site investigation of corrosion condition in concrete structures based on the *Potential Curvature Method*, a half-cell potential survey is firstly undertaken over the selected concrete surface area. To avoid missing corrosion hot spots in the selected area, the grid lines for the half-cell potential survey should have similar spacing of that of the reinforcing steel mats in the structure. The half-cell potential results are then analysed for the gradient and curvature characteristics. A computer program using a numeric approach has been developed at the ACCI for analysis of the potential gradients and curvatures. A proposed criterion to characterise the risk of active corrosion in relation with the potential gradient and curvature values is proposed and applied in the analysis.

At this stage, while further work is still needed to develop the general criteria for the correlation between the potential curvature values and the probabilities of active corrosion in concrete structures, a characteristic potential curvature value proposed by the ACCI is based on the potential gradient of 200 mV/m. Those locations with potential curvatures equal to or greater than this characteristic value are considered to have moderate to high risk of active corrosion. Such locations together with their potential curvatures are selected and plotted on a characteristic curvature contour map, which provides a view of the locations with higher corrosion risk based on the criterion. Denser potential curvature isolines on the curvature contour map indicate even higher corrosion risk at the associated locations due to their much higher potential curvature values. The *Potential Curvature Method* has been found to be very effective in the ACCI site investigation of a number of concrete structures.

Fig 1 shows the half-cell potential contour map measured over a concrete pile surface area in the ACCI investigation of a marine wharf structure. The half-cell testing was conducted along a length of 1.2 m of the pile from the tidal zone up to the joint with the deck beams. The cylindrical surface of the 850 mm diameter pile is shown in Fig 1 as a plane.

The half-cell potentials measured on this pile surface area were in the range of –376 mV to –683 mV (CSE), being more negative than –350 mV (CSE) at every single testing location. Using the ASTM C876 criteria all the reinforcing steel within this pile area would have been classified as having a high risk of active corrosion. However, there were no signs of significant deterioration from the visual inspection of the pile. It can also be seen in Fig 1 that the potential gradients are low (indicated by the widely separated contour lines) over most of this area, despite the measured high negative potential values. Further, potential values are more negative at the lower part of the pile near the water line. This phenomenon is frequently observed in the tidal zone of marine structures and generally indicates the influence of restricted oxygen access through the saturated concrete surface. In such a case, while it is obvious that an assessment according to the ASTM C876 criteria would be wrong, it is also very difficult to make a sensible assessment of the corrosion risk based on the measured potential values and the potential contour map.

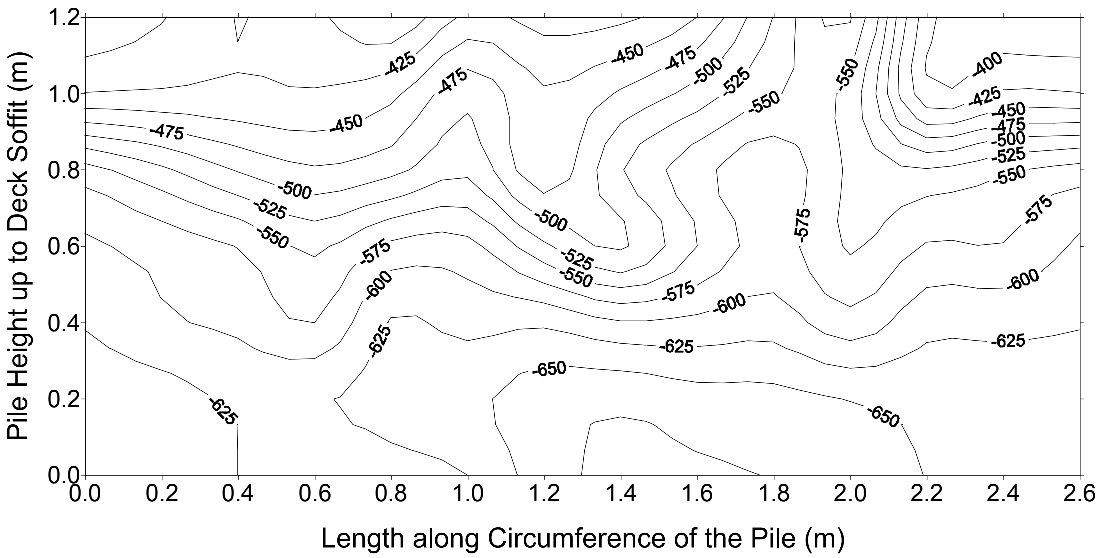


**Fig 1. Half-Cell Potential Contour Map over A Wharf Pile Surface**

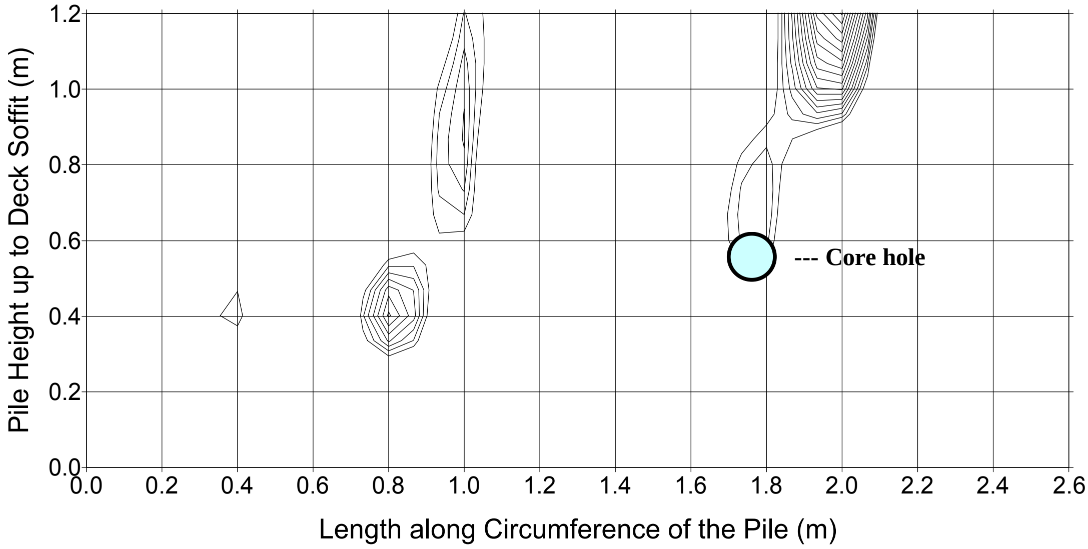


**Fig 2. Potential Curvature Contour Map over the Same Pile Surface in Fig 1**

The ACCI's *Potential Curvature Method* was applied for analysis of the potentials measured from this pile surface and Fig 2 presents the potential curvature map which identified the locations with moderate to high risk of active corrosion. Only a few locations were found to have significant corrosion risks despite the very high negative potentials measured over the whole area. On the same day of the half-cell testing on-site, a core was taken from this pile to expose the reinforcing steel for inspection.

The vertical main reinforcing bars were located with a covermeter and the core was taken on a vertical bar and by chance (as marked in Fig 2) at the lower edge of an identified location with a moderate corrosion risk.

Fig 3 is the photograph of the exposed vertical plain steel bar (32 mm in diameter) and a transverse steel bar (10 mm in diameter) at the core location, which is in the tidal zone but close to the high tide level. Surface corrosion was observed on a small local area of the vertical bar and over part of the transverse bar, while the section loss of both steel bars was not significant.

**Fig 3. Surface Corrosion Observed on the Steel Bars at A Core Location in Fig 2**

The condition of the exposed reinforcing steel in Fig 3 is consistent with the location identified by the *Potential Curvature Method*. It was also found that the high potential values observed in Fig 2 at x = 2 m, y = 1.2 m correlated with the corrosion of the reinforcing steel in the soffit of the beam intersecting the pile at this location. Other examples of a consistent correlation between the identified locations from the *Potential Curvature Method* and the exposed rebar condition are presented in another paper of this proceeding (Marosszeky, 2002), in which the on-site examples are quoted from the maintenance investigations of the Sydney Harbour Tunnel structures.

The second example is to show the application of the *Potential Curvature Method* in the evaluation of corrosion in a heavily carbonated structure. An eighty-year old concrete slab soffit was recently investigated by the ACCI. The carbonation depth was found to be about 55 mm. This exceeded the concrete cover to the reinforcing steel of 30 mm. Of the total 256 grid locations surveyed by half-cell testing, none had a potential more negative than –350 mV (CSE), 17 locations had a potential between –200 to –350 mV, and 239 locations had a potential more positive than –200 mV. The average potential of all the surveyed locations was –105 mV. Fig 4 shows the half-cell potential contour map of this slab soffit area.

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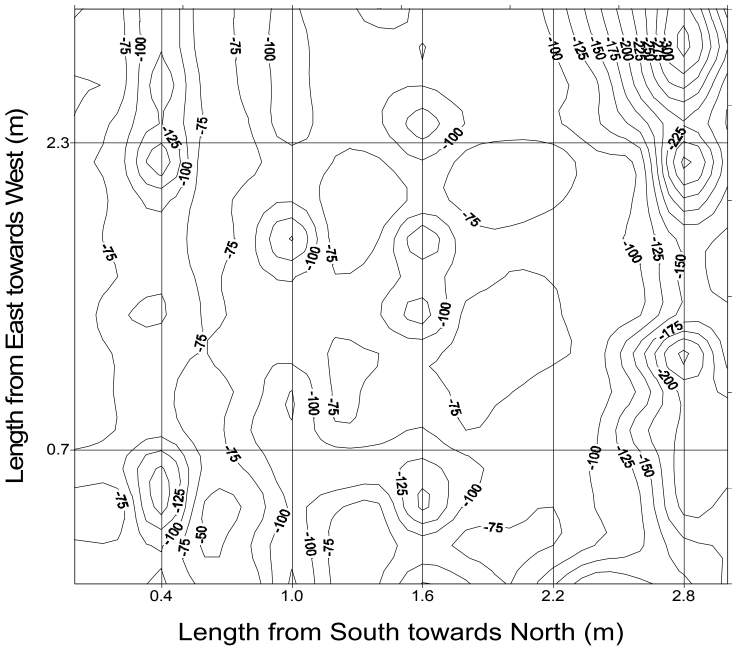


**Fig 4. Half-cell Potential Map over a Slab Soffit Area**

According to the ASTM C876 criteria, only some of those locations in the top-right corner in Fig 4 (between x= 2.5 m and x= 3 m and above y= 1 m) would have a 50% probability of active corrosion. The half-cell potentials in the rest of this soffit area were actually all more positive than –152 mV (CSE). However, cracking of the concrete cover was found along the reinforcing steel at locations with the x-coordinates of 0.4, 1.0, 1.6, 2.2 and 2.8 m. Further analysis of the half-cell potential results with the *Potential Curvature Method* was carried out and the potential curvature map was plotted as shown in Fig 5.

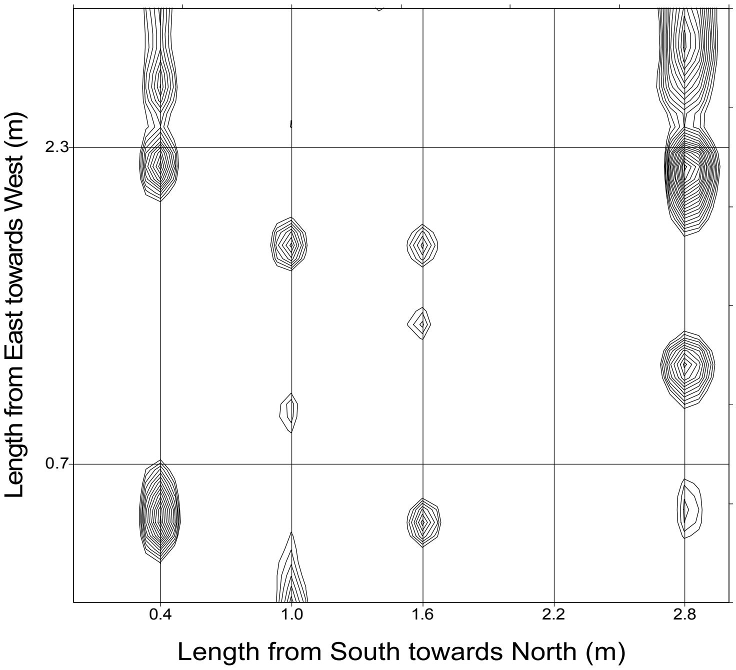


**Fig 5. Potential Curvature Map over the Same Slab Soffit Area in Fig 4**

It is clearly shown in Fig 5 that, despite all the potentials measured between the x-coordinate of 0 to 2.5 m being more positive than –152 mV (SCE), several corrosion hot spots with high potential curvatures were identified and they were correlated well with the cracks along the reinforcing steel. The reinforcing steel was exposed at one location with a high potential curvature value at the top-right corner in Fig 5 and significant corrosion was observed after removing the concrete cover.

The above examples demonstrate the advantages of the *Potential Curvature Method* for on-site corrosion investigations of two typical concrete structures; one in the splash or tidal zone and the other significantly affected by carbonation. This new approach has been found to be an effective tool for more precisely locating corrosion hot spots in concrete structures. The correct diagnosis of active corrosion locations will greatly assist further evaluations of the causes of corrosion and the implementation of effective remediation action.

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## 4 SUMMARY AND CONCLUSIONS

Half-cell potential mapping is a simple and widely used technique for the on-site evaluation of corrosion risk in concrete structures. The ASTM C876 criteria are empirically developed based on mainly the investigation of highway bridge decks affected by de-icing salts. It is known that the fundamental nature of the half-cell potential value can be affected by the chemistry of the concrete pore water at the steel-concrete interface, regardless of any corrosion activity. This questions the validity of the ASTM criteria for assessing corrosion activity based on the potential values. It has been found by others and the ACCI that these criteria can be misleading when used for corrosion assessment in concrete structures in marine environments or where they are significantly affected by carbonation. The limitations of the ASTM C876 criteria have been recognised since early 1980's. However, these criteria are still frequently used today, as no appropriate alternative approaches are available for practical engineering application.

The idea of assessing corrosion activity based on local potential gradients has long been proposed as a better method for corrosion risk evaluation. However, no practical methods have been developed to support such an approach. Further, only those locations associated with high potential gradients as well as more negative potentials relate to anodic corrosion sites.

Based on experience from site investigations and the analysis of the features of more anodic locations within the potential field, the *Potential Curvature Method* has been developed at the ACCI for identification of corrosion hot spots. These are characterised by higher positive potential curvature values, in other words, by higher potential gradients together with more negative potentials.

The *Potential Curvature Method* has been successfully used in the ACCI site investigation of different concrete structures. The method has been shown to be an effective tool for corrosion risk evaluation, regardless of the environment or cause of corrosion. It is anticipated that this new approach will enable more precise diagnoses of corrosion in concrete structures. Hence it will assist in the evaluation of the causes of corrosion and the development of effective remediation strategies.

The new approach can be used to replace the ASTM C876 criteria for corrosion risk assessment based on the results of half-cell potential surveys. Further work is being undertaken to develop the general criteria for the correlation between the potential curvature values and the probabilities of active corrosion in concrete structures.

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# MATERIAL AND ENERGY USE IN BUILDINGS

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**Summary: The climate in Iceland is temperate, wet, windy and houses are required to be designed for earthquakes. The climate is difficult for all kinds of building materials and the total load on structures from environment is high. The energy used in the occupancy phase of the buildings is mainly environmentally friendly and sustainable (geothermal heat and hydroelectric energy), but the building materials in Iceland are all imported except cement, gravel (ballast materials) and stonewool. Concrete is a very popular building material and the building industry, therefore, uses material consuming techniques that require much transport. A study of material and energy use is needed for evaluation of how sustainable the building industry is compared with other countries, and to sort out the weak points for future design efforts. The paper presents a study of material and energy use in a typical Icelandic multifamily house from the production of building materials through construction, management and maintenance over 50 years.**

**Keywords: Building, materials, maintenance, operation, energy use.**

## 1 INTRODUCTION

The impact of the built environment on the earth's climate and resources is very great, since the building industry is the greatest user of materials and energy in the world. Buildings and structures are, in themselves, the largest fixed asset of each nation and so, the importance of an economical and sustainable building process is great (anon 1999). Interest in a holistic view of the building over the total span of design, construction and use, is growing, as is knowledge about impact of building-material production, energy-use during production, transport and use, durability and service life of single components and whole buildings. Finally, the impact of material and structural disposal is of great importance in this work. Consumer societies are beginning to be aware of the effect of their use of material and energy, and the effect this has on the resources and climate of the earth. There is a need to look critically at the efficiency or inefficiency of this use, and seek the best possible solution. As pointed out by Young and Sachs (1994), there is need of a less material-intensive economy and one possibility to provide this, is redesigning consumer products to make them lighter and more compact. This statement, of course, also applies to houses, and perhaps especially in this field, as 40% of the world resources are used in the building industry, according to the CIB Report, "Agenda 21 on sustainable construction" (1999). A study of the material-use in building industry gives a basis for work on redesigning, but as local circumstances are very different, it is likely that each nation must examine these aspects carefully from their own perspective.

In this paper, only the material-use and energy aspect of the life cycle analysis (LCA) of a building is considered, as in both cases it is relatively simple to conduct statistical comparison between countries. These factors are obviously an important part of the service-life planning of the built environment, as they form a big part of the future oblications due to maintenance and operation of the buildings.

## 2 BUILDING MARKET IN ICELAND AND THE ENVIRONMENT

Iceland is a small island in the middle of the north Atlantic Ocean, beeing actually a summit of the North Atlantic Ridge. Tremendous environmental impact results from weather and earthquakes, which demands quality building, both from a residential standpoint, and for strength against geologic activity. Foundational conditions are widely difficult, with it often being necessary to remove surface layers in order to reach strata, which will bear sufficient weight. As Icelanders number 280 000, the building market is small. Natural conditions are difficult, with limited option for production of local materials, and thererfore, imports are an important aspect. Domestic building materials are essentially only various types of fill, cement and rock wool. A quantity of raw materials is imported for further production, such as of styrofoam insulation, insulating glass and glulam. Still more materials are imported,--usually in consumer packages.

As a result, buildings are, to a large extent, constructed of heavy, domestic material (concrete) and it may be expected that transport costs are a large part of domestic expense, due to material weight, and in import costs, due to the distance from the producer and large volume of the package units.

Hydroelectric energy remains adequate for local use in Iceland, in addition to which, geothermal energy is in large surplus, so the vast majority of energy used in building is environmentally friendly. Iceland's position is, thus, unique in several respects, and there is every reason to examine how sustainable the building market is, especially concerning material-use and energy.

The aim of the study is to observe the weight of different materials and building components in the whole picture, from "craddle to grave". The knowledge acquired, makes comparison with others possible, and also adds a piece of information to the bigger puzzle of international sustainability of the built environment. The problem, when making comparison between cited cases, is that it is often difficult to see what, in fact, the figures are based upon, so the risk is that comparison is made between apples and oranges in some cases. It is of considerable scientific interest to reach an agreement as soon as possible on how such figures should be reported. It is hoped that the following information and discussion will be of some interest to others besides Icelanders.

## 3 MATERIAL AND ENERGY USE IN A MULTI-FAMILY HOUSE

### 3.1 Introduction

In order to examine material and energy use in the building industry, it was decided to consider a three-wing, multi-family dwelling with a total of 24 apartments. This building has been used to analyse building costs, and has been subsequently followed through its 10 years of use to date. As a result of its previous history, there is considerable detailed data on material and energy usage during its construction, as well as it being possible to obtain information on energy-use during its 10-year period of operation. The building is typical for a multi-family house in Iceland; the foundations are built from locally produced concrete on a rockfill; external walls and all floors are from local concrete; internal walls are either from light-weight (pumish) concrete blocks, or timber frames, covered with wood-based or gypsum boards, and finally, a timber roof, clad with corrugated steel. The house is heated with geothermal energy, (in Reykjavík sold as cubic meters of hot water) and, in the case studied, the temperature of the incoming water is about 80 °C. Electricity is produced by hydro-power stations.

In the previously discussed project, material amounts were divided into 26 building aspects, and in all, 25 different material categories. Then, the embodied energy in materials and the energy used to transport the materials to the building site, was estimated. Information about some aspects is still lacking as of this writing in August 2001, but the subsequent information will only make minor changes in the whole picture.

### 3.2 Use of building materials in a new building and in maintenance

The building can be divided up in three individual parts and the results presented are for one third of the whole structure.

Key figures for the size of this part of the house and the building lot are shown in table 1.

**Table 1. Size of building (incl. garage for four cars) and building lot**

| | | |
|---|---|---|
| Number of apartments | 8 | |
| Area of ground floor (gross area) | 341.3 | $m^2$ |
| Total floor area (gross area, walls included) | 1171.0 | $m^2$ |
| Total usable floor area (netto) | 998.7 | $m^2$ |
| Area of building lot (incl. car parking) | 1666.0 | $m^2$ |

The material use in the construction phase is calculated from real figures for the house, the maintenance is then estimated, based on the estimated maintenance of the building during a total time span of 50 years.

Techniques involved in major maintenance or refurbishments are generally of two types. On the one hand, projects dealing with the whole building are undertaken at a predetermined time. On the other hand, projects are undertaken as, and wherever, the need becomes apparent. The prior option offers more operational reliability, though sometimes things are replaced before their useful service life has expired. The second option means the repair periods will be more frequent, with less reliability and that material service life is stretched to the maximum. Whichever option is chosen, depends upon the type of house/building and the wishes of the owner/operators. In the case of residential buildings, the latter option is often chosen, and is the one on which the following discussion is based.

When calculating the quantity of material-use, in connection with maintenance and refurbishment, it is easy to decide the effect of some efforts, such as repainting, done fairly often over the time span considered (50 years). In these cases, the major maintenance or refurbishment can be calculated as occuring at one instance in time, and the average service life is used without a great loss of accuracy, Fig. 1a. In other cases, when the distribution in service life is great for the same type of component, and the average service life is of the same order of magnitude as the time span considered, great care must be used in the calculations. In such cases, the frequency distribution often "straddles" the end point of the time span, Fig. 1b, and the actual maintenance or refurbishment done is decided by the average service life and the shape of the curve. For the distribution, D1 in Fig. 1b, the maintenance is almost completely inside the time period, but for distribution, D2 only a little more than half of the components have been maintained inside the time limit. To ensure a sufficiently exact solution, the calculations should take into account the actual distribution in service lives. This is especially valid if maintainance efforts needed are different, and the choice is dependant on the age of the component. The necessary information about different materials is still very unsatisfactory, at least that is the case in Iceland, and the actual frequency distributions for service lives of the materials concerned are not known. Therefore, an estimated maintenance interval and proportion of maintenance, as a ratio of the initial material use in the component, is used to allow an initial estimate of the effect of maintenance on material use. For example, the effect of refurbishment of windows with a average service life of 50 years is decided as 50% of the initial material quantity, at the end of the calculation perid of 50 years, half have been refurbished, the other half is still functional.

**Figure 1. The effect of maintenance intervals, shape of service life distributions, and the total timespan used in calculations**

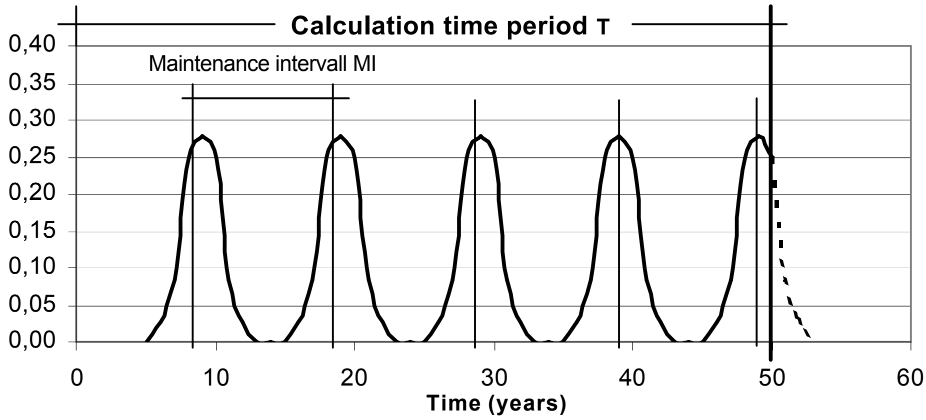


**Fig 1a Many maintenance efforts with short intervals and tight distributions**

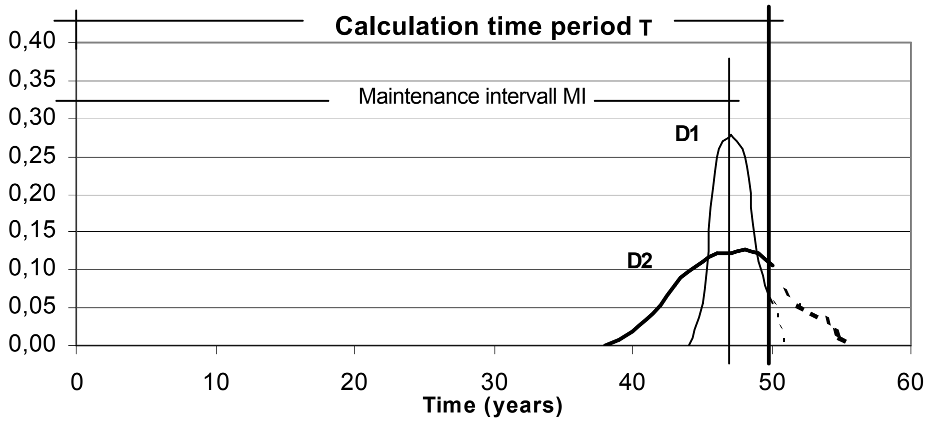


**Fig 1b One maintenance effort: examples on two different shapes of distributions**

The estimated maintenance interval and proportion of maintenance for materials and components is shown in Table 2. Based on these figures (and the initial material use), the quantity of building materials (mq) in maintenance are calculated as a linear ratio betweeen the total time(T) span and the average maintenance interval (ami), multiplied by the material use (mu) and the proportional factor (pf);

$$mq = (T/ami) \bullet mu \bullet pf.$$

Material-use for the new building as a total mass for the different building parts and also calculated per unit area of usable floor is shown in Table 3. The table also shows the estimated material use for maintenance.

**Table 2. Maintenance intervals for different components and surfaces.**

| Effort | | Average maintenance interval (years) | Proportion of initial amount |
|---|---|---|---|
| Repainting | wood, outdoors | 4 | 1.0 |
| | concrete, outdoors | 8 | 1.0 |
| | metals, outdoors | 12 | 1.0 |
| | surfaces, indoors | 5 | 1.0 |
| Maintenance of interior surfaces | | | |
| (spackling,etc.) | walls and ceilings | 30 | 1.0 |
| | floors | 15 | 0.05 |
| Interiors | kitchen | 25 | 1.0 |
| | bathroom | 25 | 1.0 |
| | whiteware | 25 | 1.0 |
| | plaster surfaces | 25 | 1.0 |
| | carpets and linoleum | 15 | 1.0 |
| | interior fittings | 25 | 1.0 |
| Windows | glass panes | 25 | 1.0 |
| | wood | 50 | 0.5 |
| Doors | | 50 | 1.0 |
| Roofs | metal sheets | 40 | 0.5 |
| | wood (structural) | 40 | 0.15 |
| Concrete surfaces | maintenance | 8 | 0.05 |
| Technical apparatus | drain pipes | 40 | 0.2 |
| | water pipes (heating and drinking water) | 40 | 0.5 |
| | electricity | 40 | 0.5 |
| | ventilation system | 30 | 0.5 |

**Table 3. Material use in the new building and maintenance over 50 years**

| | Total weight (ton) | Weight/usable floor area (ton /$m^2$ ) |
|---|---|---|
| NEW BUILDING AND YARD | | |
| Foundations 2034.98 1.74 | | |
| Building Envelope | 996.76 | 0.85 |
| Indoor and outdoor surfaces | 325.16 | 0.28 |
| Interior fittings | 7.16 | 0.01 |
| Technical systems (incl. pipes) | 3.56 | - |
| Yard and drain pipes | 479.00 | 0.41 |
| Total - yard included | 3846.63 | 3.30 |
| Total - yard excluded | | 2.89 |
| MAINTENANCE OVER 50 YEARS | 112.94 | 0.10 |
| Material weight per ground floor area | 12.11 | ton /$m^2$ yard excluded |
| Total material weight per yard area | 1.90 | ton /$m^2$ yard included |

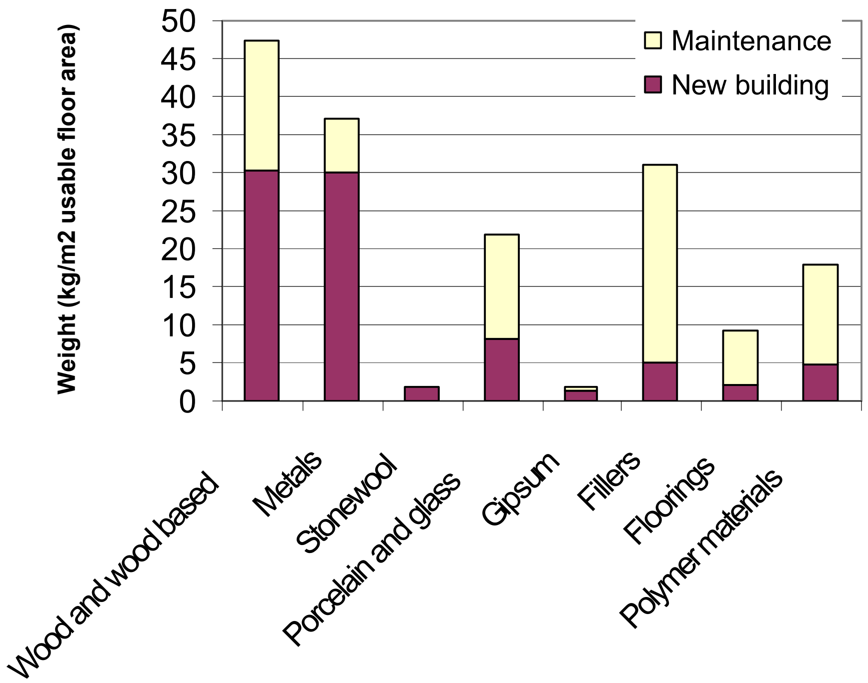


**Figure 2. Use of different materials in the building per $m^2$ usable floor area**

Concrete, cement and various earth materials = 3212 kg/$m^2$ usable floor area

The material quantity in the construction phase and maintenance, shown in Table 3 and Figure 2, is fairly exact as it is based on information from the construction of a real house, and includes all spillage of materials. The total weight of the house is impressive, to say the least, the calculated weight of 2.89 ton/$m^2$ usable floor area. This is very high compared with figures seen in the literature, from Norén (1999) a figure of 578 kg/$m^2$ usable floor area is found for a wooden house on concrete foundations. It is of interest to note that for some of the materials, the quantity used during maintainance period of 50 years will be considerably more than was the case for the new building.

If weight figures presented in Table 3 are typical for the city of Reykjavik, then it is clear that local pressure on the earth's mantle, caused by the built environment, is considerable. To make an comparison, it has been estimated that at the last Ice Age, the thickness of the glacier cap over Iceland was from 500 m (at the coast) to 1500 m (inland). The resulting height change amounted to 40 - 110m lower surface elevation of land with respect to sea level, than presently. This was due to combined effect from the crush of the ice and lower sea surface because of less water in the oceans. An ice-cap with a thicness of 500 m has an approximate weight of 450 ton/m$^2$ and the crush from the built environment is about 1.9/450 or 0.4% of this. The latter should, therefore, in time have a measureable effect on the land elevation. Exactly how much is complicated to estimate, but it is greater than results from a straight-forward calculation, based on the above figures, gives. The effects of the activities of man on the earth and its environment are more complicated than most people care to admit!

The main reason for the high material-use in the new building is the great use of rock fill material in foundations and the parking lot. The material-use in maintenance and refurbishment is very modest compered to that used in the initial construction phase, this is mainly due to the fact that the large masses of the building have a longer service life than the timespan considered in the calculations.

### 3.3 Energy in materials, transport and operation

The embodied energy in materials and energy used in transportation of materials is estimated from material-use in the construction phase and maintenance, Table 4. Figures of specific embodied energy found in the literature are used for all imported materials and cement, Icelandic figures were calculated for domestic materials, such as rock fills and rock wool. The figures for embodied energy vary widely from source to source, as known, and thus must be considered with some care. In the calculations, it is generally assumed that energy use due to a 500 km transport to an export harbour must added published figures on embodied energy . Figures for transportation of materials are based on personal information from Icelandic transport companies.

**Table 4 Energy use in building; embodied energy, transport and operation**

| | GJ | GJ/m2 |
|---|---|---|
| NEW BUILDING | | (floor area) |
| Embodied energy in materials | 4181.76 | 3.583 |
| Transport of materials | 468.52 | 0.401 |
| Energy used on building site | | |
| oil | 0 | 0 |
| electricity | 123.88 | 0.106 |
| geothermal energy | 1014.89 | 0.870 |
| **Total for new building** | 5789.04 | **4.961** |
| MAINTENANCE 50 years | | |
| Embodied energy in materials | 2558.46 | 2.192 |
| Transport of materials | 369.73 | 0.317 |
| Total for maintenance | | **2.509** |
| ENERGY USE IN OPERATION | **GJ/year** | **GJ/m2,year** |
| electricity | 89.15 | 0.076 |
| geothermal energy | 742.95 | 0.637 |
| **Total for operation pr. year** | | **0.713** |

Embodied energy in the house from the start, and energy used during the construction phase is a total of 4.961 GJ/m$^2$ usable floor area, which can be compared with figures in the literature; Johnston and Verbeek (1998) give the figure 2.28 for a "greenhouse" and 2.7 GJ/m$^2$ in a more traditional house and Norén (1999) provides a figure of 1.49 GJ/m$^2$ floor area for a wooden house in Scandinavia. The energy content in the Icelandic building is high in the comparison, mainly due to the very massive construction that is traditional in Iceland and high energy use for heating during the building period.

Energy in transportation is a little higher than 1/10 of the embodied material energy, but much less than the energy used on the building site, the latter being mainly due to the need to warm up the indoor working area during winter, and then to dry out building moisture before the building is handed over to the user. Energy in connection with maintenance of buildings, both the embodied material energy and trannsport energy, over the time perid of 50 years, is a total of 2.509 GJ/m$^2$ floor area, or about 50% of the total embodied energy in the building from the start.

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The annual energy used for heating and electricity is a total of 0.710 GJ/$m^2$, floor area. Therefore, operational energy over a period of less than seven years equals the total of initial embodied energy of the house and energy used during the building phase.

As previously mentioned, heating of the completed building is obtained from geothermal water at an incoming temperature of about 80°C. How effectively this hot water energy is used depends upon the type of heating system and building insulation, but in general the outgoing water temperatures may be estimated at 30-35°C, depending on air temperature. The energy for heating the house is, therefore, of a low quality (low exergy). Of the total heating energy used, Q, the exergy content is only 0.21Q. The energy used in production of insulation materials is (almost) totally high quality energy from electricity or fossil fuels. In this context, it is interesting to consider whether insulation is as energy efficient as commonly held, considering it is manufactured using energy of so much higher quality than is saved by the insulation. When this is examined for Iceland, it emerges that even there, it is energy-efficient to insulate buildings thoroughly. (The building regulation requires for walls: U-value = 0.3 - 0.4; and for roofs; U-value = 0.2 W/$m^2$K). The total energy saved by good insulation equals the embodied energy in the produced insulation after only 0.06 years and even the exergy part of the low quality geothermal energy saved balances that of the insulation in 1.2 years (Marteinsson, unpublished paper).

The building mass in Reykjavík is young--the average age being less than 30 years--and the re-use of building materials is in the beginning phase. The energy in connection with tearing down of structures and re-use or disposal of materials has not yet been considered.

## 4 CONCLUSIONS

It is evident that the traditional building technology in Iceland is far from material-efficient, and the energy emboided in materials, and used for transporting the materials to the building site, is high. The greatest part of the excessive weight of the houses is, though, in earth materials that have, as such, little emboided energy and could be efficiently re-used when the service life of the building is over. However, there is every reason to limit excavation of materials as much as possible, since excessive quarrying uses high-quality, imported energy, and has a negative impact on the environment in all aspects. Limiting excavations automatically limits energy used in construction. Reduction of material mass is most likely possible through changes in building methods regarding foundations and more material efficiency in the structural concrete used. The transport energy will then also decline, but the nation will always be very dependant on imports, which results in rather energy-consuming transport . Reduction of energy used during construction must also be considered, and more use of weather protection during the construction phase may be one solution.

The experience of reuse of materials in Iceland is very limited to date, but this will probably change quickly, especially since the material mass in the buildings is high, as previously discussed. Information about the energy cost of tearing down obsolete buildings and the accompanying re- use, or disposal of the the materials will be gathered at later stages in the project.

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# A Framework For Service Life Design Of Concrete Structures

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**Summary: BRE are currently developing a generic service life design system which will provide specific guidance for concrete structures. This paper describes the work that has been carried out to date in the development of the overall framework of the design system. This framework will be expanded as the project progresses to provide targeted guidance and tools to assist the practising engineer in achieving a structure that is durable for its required service life and in optimising whole life costs.**

**Keywords: Service life, holistic design, concrete**

## 1 INTRODUCTION

There are many factors in the design and construction process that can determine whether a structure will meet its design service life. There are numerous examples of problems that have occurred where structures have been exposed to aggressive environments, where design and workmanship have been poor, and where communication between the parties involved in the design and construction process has been inadequate (Jones *et al*. 1997). Problems have also arisen as a result of a design philosophy in which minimising initial costs was paramount. Premature and costly repair, remediation or even replacement may be required. These problems highlight the need for:

- A holistic approach to design embracing the entire construction process and explicitly addressing service life (Somerville (1999), Quillin (2001), Jones et al. (1997)Edvardsen & Mohr (1999)).
- Whole life costing in assessing the cost performance of constructed work and in deciding between alternative means of achieving the client's objectives (Clift & Bourke (1999), Somerville (1995)).

BRE are currently developing a service life design system that will provide guidance and tools to asist in the design of concrete structures. The key elements of the design system are listed below and reflect the fact that durability problems can arise as a result of failure at any one of a number of stages throughout the construction process:

- Client brief
- Assessment of environmental loads
- Definition of required performance under environmental loads
- Conceptual design
- Detailed design
- Execution
- Maintenance and management in use

The main interactions between these elements are illustrated in a schematic diagram of the service life design process in Figure 1.

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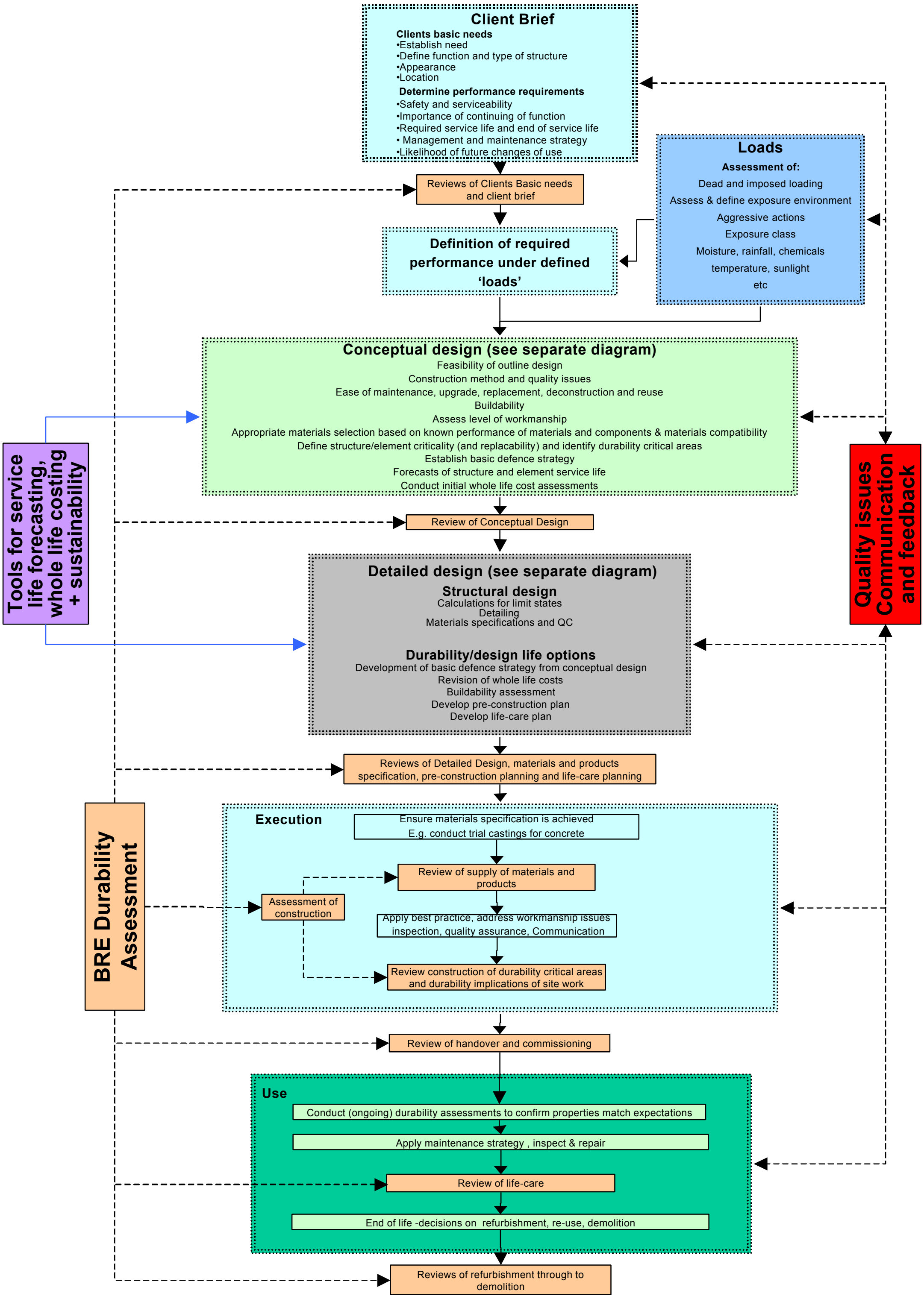


**Figure 1. Schematic diagram for the service life design process**

Figure 1 puts a strong emphasis on the Client brief and Conceptual design stages, whilst embracing structural design, and permitting the introduction of the BRE Durability Assessment procedure (Hooper *et al.* 2002). Execution, construction quality and use are crucial if the intended performance is to be achieved in practice and should be considered at the Conceptual design stage. The need for good communication up and down the supply chain is also highlighted. The design system at this level is generic and is compatible with ISO 15686-1, Buildings and constructed assets, Part 1: General Principles (published in the UK as BS ISO 15686-1). However, within the individual elements the system is being developed with concrete as the primary focus and will build on existing codes and standards, as well as other relevant guidance documents.

## 2 CLIENT BRIEF

The Client brief addresses the relevant needs and aims of the construction project, resources to be provided by the client, the details of the project and any appropriate design requirements within which all subsequent briefing (when needed) and designing can take place. This stage of the design system should highlight the important issues for the client (and parties advising and assisting the client in the preparation of the brief) to address in order to enable effective service life design. It has been sub-divided into 2 parts; the clients basic needs and the consequent performance requirements.

A schematic diagram for the Client brief, is shown in Figure 2. The diagram also covers two other elements of the design system; the ‘assessment of environmental loads’ and the ‘statement of required performance under defined loads’. These are briefly discussed below.

### 2.1 Clients basic needs

The client needs to consider a number of basic issues that can influence the design and the performance requirements for the structure. These include the following:

- The type of structure and its location.
- The planned function(s) of the structure and its component parts.
- Any appearance/aesthetic requirements (initially and throughout the life of the structure).
- Requirements for usable space, dimensions, services and fittings.
- The period of tenure and the requirements for the structure at the end of this period.
- Future changes of use to increase flexibility and minimise the risk of obsolescence.
- Restrictions to the design (e.g. planning regulations or desired sustainability credentials).

### 2.2 Performance requirements

The client’s basic needs addressed above will enable a number of performance characteristics that the structure will need to meet to be specified. These performance requirements, together with an assessment of the environmental loads, will form the basis of the design process and include:

- Safety and serviceability.
- Service life and what constitutes the end of service life.
- Importance of continuity of function and flexibility to accommodate changes of use.
- Management and maintenance requirements.
- Acceptable level of inspection, maintenance, repair or replacement.
- Acceptable costs (capital and operating costs).
- Design restrictions.
- Agreed sustainability credentials.

## 3 ASSESSMENT OF ENVIRONMENTAL LOADS

The environment to which the concrete will be exposed is a key factor in designing for a given service life. Relevant factors include:

- The general environment conditions (***macro-climate***)
- The specific location and orientation of the concrete surface being considered and its exposure to prevailing winds and rainfall (***meso-climate***),
- Localised conditions (***micro-climate***) such as surface ponding, exposure to surface run-off and spray, aggressive agents, regular wetting, condensation etc. including those arising as a consequence of the interaction between the structure and the environment (e.g. cladding to keep the structure dry or ponding due to poor detailing etc.).

## 4 REQUIRED PERFORMANCE UNDER DEFINED ENVIRONMENTAL LOADS

In the schematic diagram for the overall framework (Figure 1) the assessment of the environmental loads are brought together with outputs from the Client brief to provide a definition of the required performance under these loads. This then forms the starting point for the durability-related aspects of the Conceptual design stage in which the basic defence strategy is formulated (see below).

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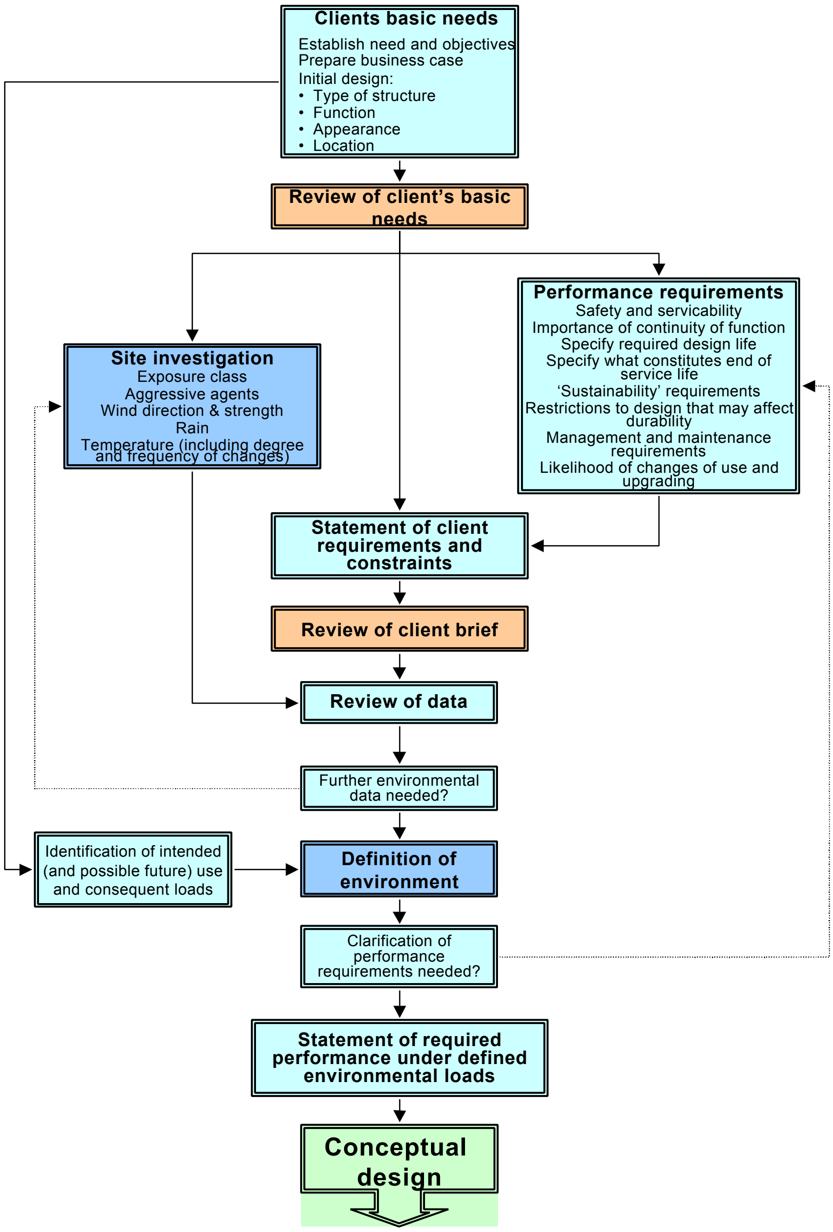


**Figure 2. Schematic diagram for Client brief**

## 5 CONCEPTUAL DESIGN

Conceptual design forms the core of the service life design process. Based on an assessment of loads and of the clients requirements key decisions are made regarding how best to resist the environment (see 'Basic defence strategies below) and the choice of structural form to minimise deterioration (the aim should be to minimise the effects of water, which are influenced by wind and temperature, by ensuring that water drains off the structure quickly and reducing penetration at durability-critical areas). For routine structures this may be straightforward, involving some combination of appropriate prescriptions with good detailing practices. In more extreme situations greater rigour may be needed, using either a design out approach, or a multi-layer protection system - or a combination of the two.

The likely level of workmanship and materials quality need to be taken into account. Decisions made at the conceptual design stage will also influence the maintenance and management strategies. Parts of the structure that may be vulnerable to deterioration need to be identified and their criticality and replacability (i.e. life long, repairable, maintainable) defined. Whole life cost assessments of potential design options may be carried out prior to selection of the most appropriate option. These assessments may require initial forecasts of component service lives in their specific exposure environment (ISO-15686-1, ACI (2000), Quillin (2001),), Sarja A & Vesikari E (1996)). These may be carried out in more depth at the Detailed design stage.

The ease with which a design can be translated into reality (i.e. its 'buildability') needs to be assessed throughout the design process. Structures and details that are difficult to build may, in practice be constructed to a lower quality than those that are easier to build with consequences for durability. The construction team needs to be able to comment on the buildability of the design and have a positive input. It is also important for the designer to be given access to preferred construction methods and know their advantages and constraints (CIRIA 2000).

A schematic diagram showing the key (durability-related) stages of Conceptual design and their interactions are given in Figure 3. Basic defence strategies (i.e. 'design out' and 'provide resistance'), the use of whole life costing in assessing alternative strategies, and management and maintenance strategies are discussed below.

### 5.1 Selection of basic defence strategy

The selection of the basic defence strategy during the Conceptual design stage is one of the key decisions affecting durability. Schematic diagrams for the selection of appropriate basic defence strategies are given in Figures 4 and 5. There are 2 main approaches that can be applied (Somerville (1999)) and, depending on the circumstances, there are a number of options within each:

1. Design out
    - Exclusion of water (e.g. enveloping cladding system, continuous structures avoid joints)
    - Use of non-reactive materials (e.g. non-ferrous reinforcement, sulfate-resisting cements)
    - Cathodic prevention
2. Provide resistance
    - Single-layer protection (depth of cover, use of low permeability materials)
    - Multi-layer protection (coatings, corrosion inhibitors etc in addition to good quality concrete and appropriate cover depths etc)

Figures 4 and 5 show an approach to selecting an appropriate basic defence strategy based on the environment, nature and criticality of the structure/element, quality of materials and workmanship etc. Figure 5 shows that the process of developing the appropriate defence strategy is split between the Conceptual and Detailed design elements of the service life design system.

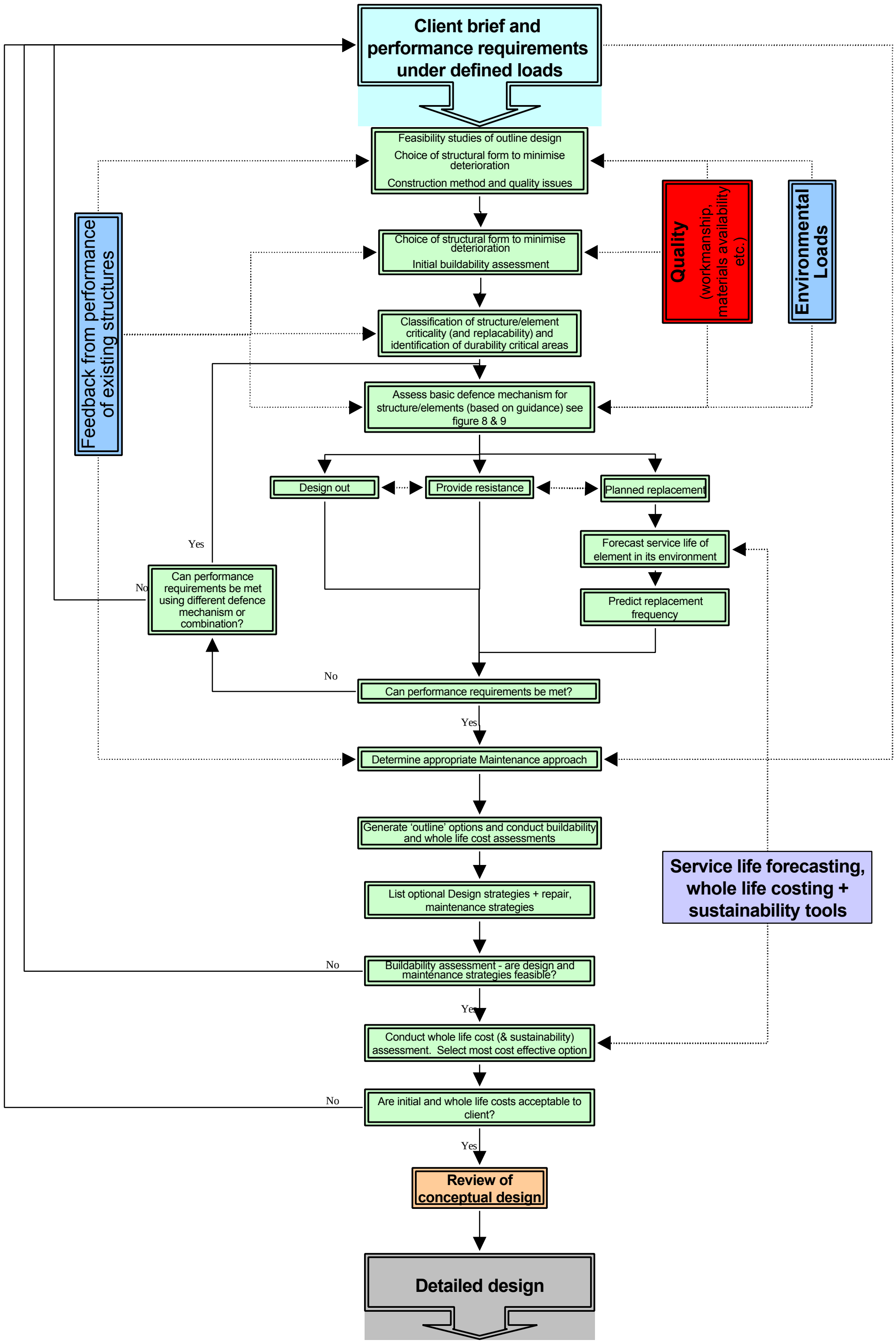


**Figure 3. Schematic diagram for Conceptual design**

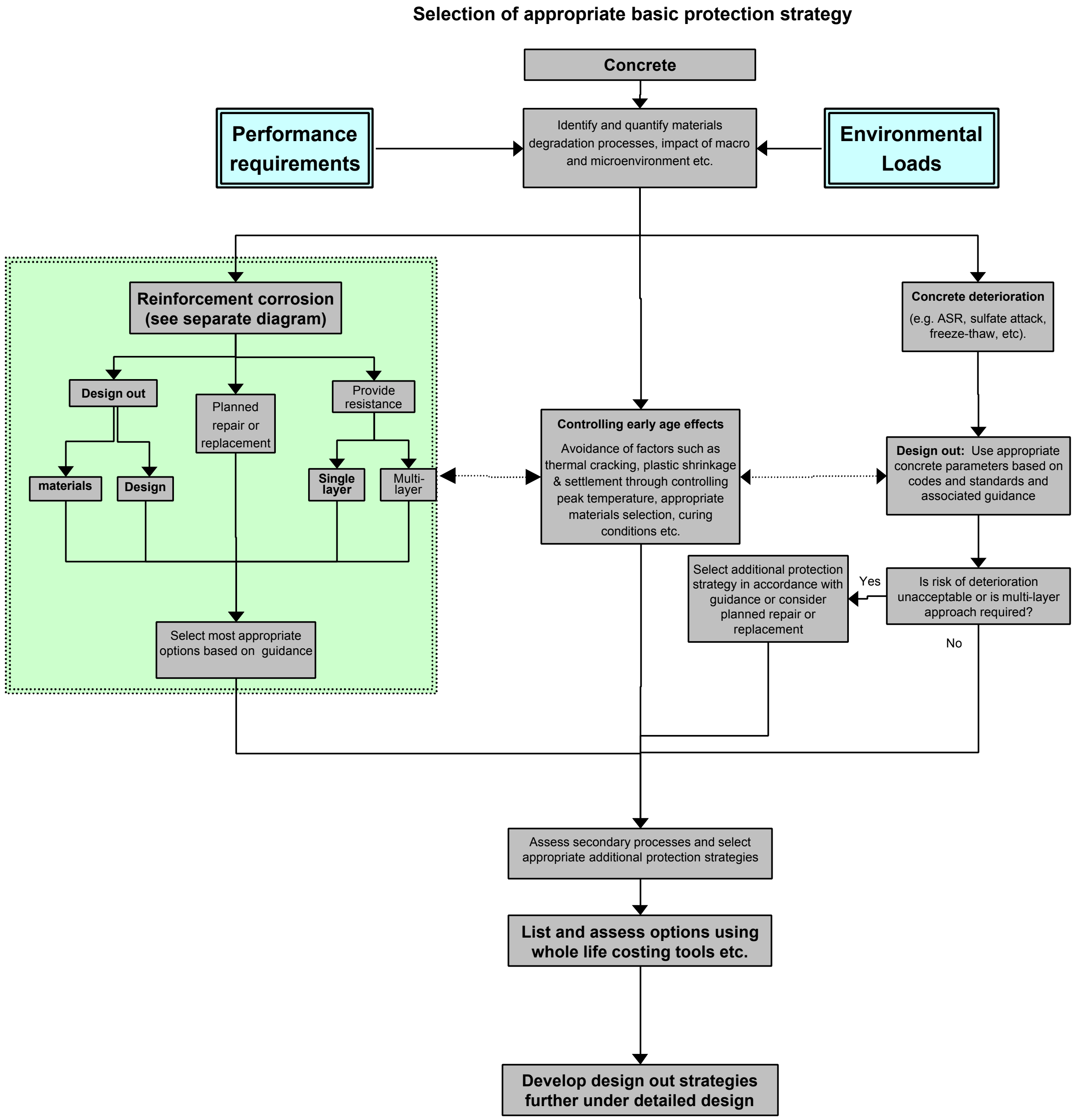


**Figure 4. Selection of appropriate basic defence strategy**

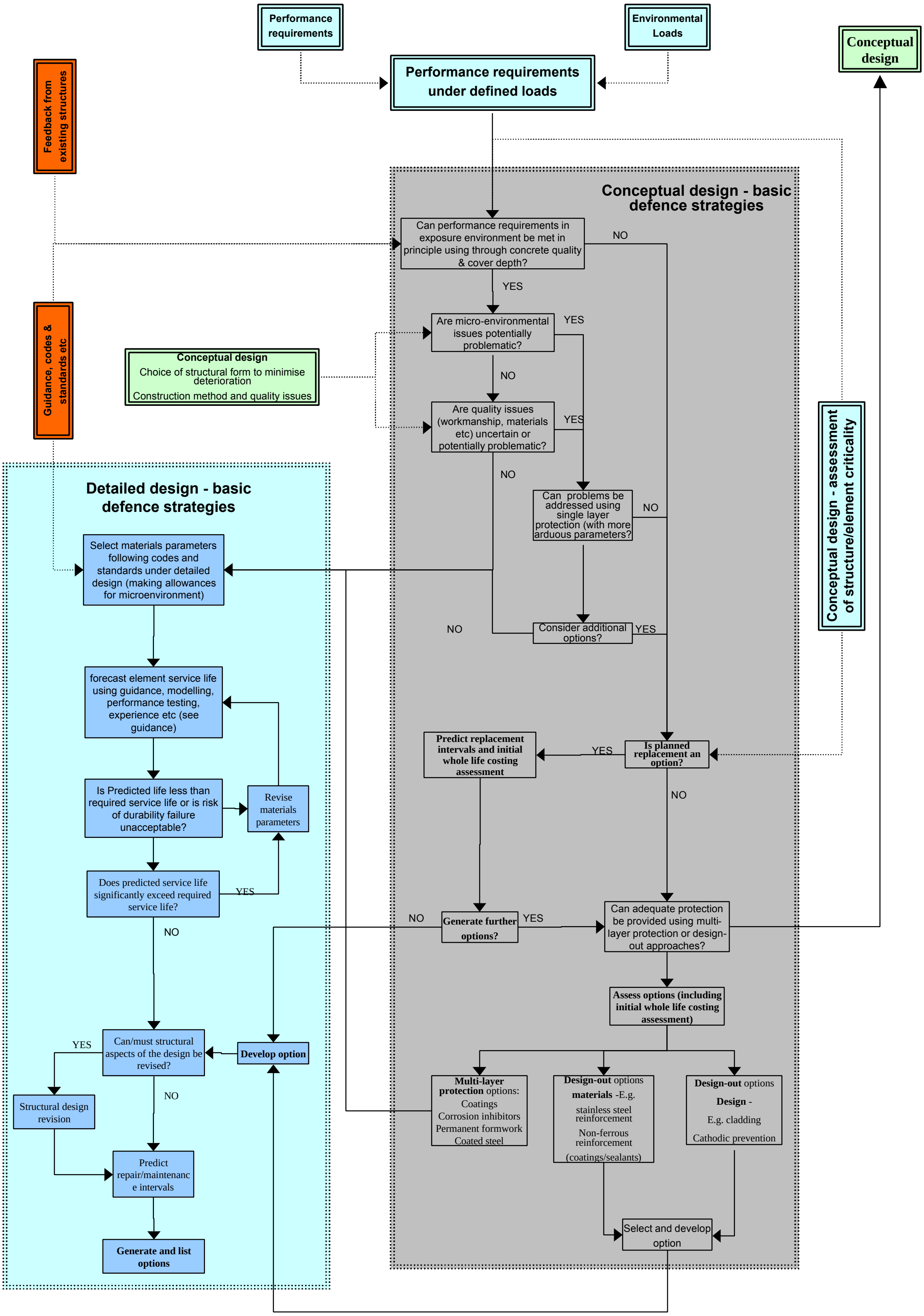


**Figure 5. Selection of basic defence strategy for reinforced concrete**

## 6 DETAILED DESIGN

In this stage, the Conceptual design is put alongside conventional structural design, and developed into something that can be built. Conventional structural design is essentially a numerical process involving structural analysis and section design, whilst satisfying prescribed limit states. The following factors will be included in the output from structural design. These will also have a bearing on Detailed design for service life:

(a) Appropriate concrete cover to provide bond and fire resistance
(b) A requirement for concrete strength grade
(c) Reinforcement arrangements to limit flexural cracking under dead or imposed loads
(d) Shapes and sizes of components, to limit deflections, and provide stiffness and stability
(e) Consideration of movement, under the effects of temperature and wind

Of these, (a) and (b) will require direct examination for service life design, when following a "provide resistance" strategy. For (c), the influence of the design crack widths may also require consideration, and a check made that the proposed rebar arrangements are practical in terms of the anticipated level of workmanship. For (d), size and shape are again important, in terms of influence quality of construction and buildability. However, when considered alongside (e), for facades and exposed surfaces generally, they are of great importance, in affecting moisture transportation, due to the combined effects of water, temperature and wind. The need for good structural detailing, with ingress of water very much in mind, needs to be emphisised.

Any areas that are critical to durability and that require particular attention need to be identified and addressed during the Detailed design stage. These should be as easy to build as possible. Minor elements with short-term service lives need to be easy to replace during the lifetime of the building or structure. Inspection, maintenance, repair and replacement plans should also be developed at this stage. This plan serves to ensure that the performance of the structure matches that required by the client.

Against this background, a schematic diagram for Detailed design is shown in Figure 6. This shows the basic principles and scope. However, aspects of Detailed design are also addressed in Figure 5.

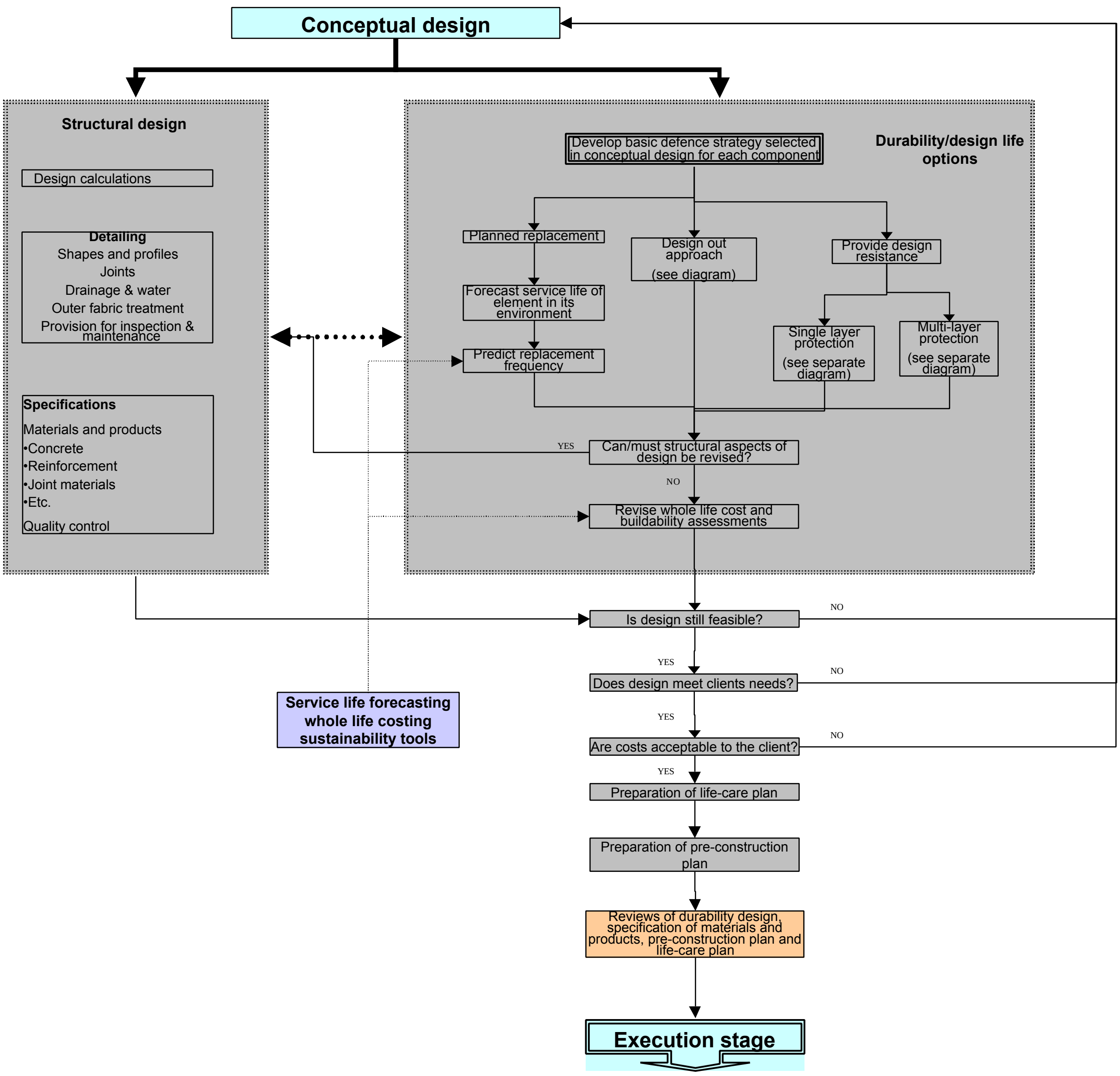


**Figure 6. Schematic diagram for Detailed design**

## 7 EXECUTION

The execution phase will be crucial in determining whether the service life of the structure will meet that included in the design. There have been numerous examples of durability problems due to poor concrete, inadequate cover to reinforcement, chlorides in the concrete or combinations of these (Jones *et al.* 1997). These have led to various forms of corrosion induced damage, such as cracking and spalling of concrete, reductions in structural capacity, as well as creating the need for repair works and introducing a larger than anticipated maintenance burden.

The quality of workmanship is influenced by the quality of design and detailing. Some designs are difficult to build, even with reasonable standards of workmanship on site. Cover, curing, compaction, supervision and communication also influence quality.

There are two (compatible) basic ways of addressing construction quality and variability, particularly in providing resistance to corrosion (Somerville 1999):

1) Raise standards over time to an acceptable and repeatable level by encouraging better design and detailing (buildability), improving communications from the designer to the site operative, improved education and training, benchmarking best practice, and the use of method statements and certification. This option involves improvement, via the development of a quality package, within a systems approach. It is aimed at raising standards to a known acceptable (and repeatable) level over a period of time. Design plays an important role through the concept of buildability.
2) Assume current variability and standards and consequently rely on strategies such as multi-layer protection systems and the use of 'design out' approaches.

Option 2 recognises that Option 1 cannot be widely achieved at present and seeks to address the variability in quality that currently exists. The design system will, therefore, be developed with current variability in mind. However, to meet any given set of performance requirements, a cost penalty will necessarily be incurred.

## 8 MAINTENANCE AND MANAGEMENT IN USE

If the structure is to meet the performance requirements it is necessary to ensure that the structure is being used in a way that is compatible with the design intent and that inspection, maintenance, repair and replacement are conducted in accordance with the plans developed under the Detailed design stage. End of life decisions, such as the implications of obsolescence possible changes of use and the loss of fitness for purpose also need to be addressed. These issues will influence the design and so must also be considered as part of the client brief and Conceptual design.

## 9 QUALITY, COMMUNICATION AND FEEDBACK

Quality, communication and feedback are key issues in ensuring that structures meet their required service lives. The importance of these issues is illustrated in Figure 1. Particular issues include:

- The importance of communication throughout the construction process.
- Raising standards over time.
- Assessments of buildability.
- Extent to which the design meets the client brief.
- Extent to which the actual work meets the design.
- Quality of materials supply.
- Availability of skilled workers.

## 10 CONCLUSIONS AND NEXT STEPS

This paper summarises the main components of a proposed service life design framework. The basic principles of the framework have been established, and the key elements identified. The framework will be developed into a full design system providing tools and guidance to support service life design and the optimisation of whole life costs for concrete structures. The level of detail required at each stage will depend on the particular element of the design system that is being addressed. Deterioration processes which directly affect the concrete can be readily addressed via links to relevant codes, standards and other consensus guidance documents. However, there are other aspects that are less well covered in the literature and where the level of detail to be included is more of an issue. These include:

- Good detailing practices for facades, outer surfaces, waterproofing, drainage etc.
- The coverage of design out options, such as:
- Non-degradable materials
- Provision of effective physical barriers.

A case could be made for following up on the generic framework and elements included here, by making specific recommendations, in principle and detail, for different types of structure in different environments. As an example, a "design out" approach could be recommended for buildings where an enveloping cladding system could be provided. Highway structures are another matter, with different problems requiring different solutions.

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The design system, in the short term at least, assumes the current levels of variability during the execution stage and in the quality of construction. In the longer term the question of how best to move on from present practices needs to be addressed. The holistic service life design framework proposed here could fall on stony ground unless some indication is given on how best to move from present practices and prescriptions to bring these proposals into effect. Such a move would almost certainly require changes to how the supply chain operates at present.

## 11 ACKNOWLEDGEMENTS

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# Sydney Harbour Tunnel – Technical Aspects Of Asset Maintenance Strategies For Long Term Serviceability

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**Summary: This paper examines techniques used in the Sydney Harbour Tunnel Asset Protection program designed to ensure the tunnel's long-term serviceability. A comprehensive maintenance program was put into place shortly after the completion of construction. State-of-the-art techniques are being employed and developed for investigations and condition monitoring. A new analytical approach has been developed for more appropriate interpretation of half-cell potential test results instead of merely relying on the criteria contained in ASTM C876. Another technique, the injection of micro-cement grouts into concrete cracks, is currently being investigated for corrosion control. Laboratory research is also being undertaken to study the influence of cracks on the corrosion of steel reinforcement in submerged marine structures. The cooperative effort of the consulting and maintenance teams combined with the use and development of advanced techniques in an ongoing maintenance program has played an important role in the tunnel's asset maintenance strategy.**

**Keywords: Maintenance, Tunnel, Marine environment, Concrete, Corrosion**

## 1 INTRODUCTION

The Sydney Harbour Tunnel (SHT) is one of the largest privately funded infrastructure projects built in Australia. Since the tunnel's official opening in August 1992, a total of 222 million vehicles have used the tunnel. It is expected that 34 million vehicles will drive through the tunnel in 2001: about 93 thousand vehicles per day. The tunnel plays an important role in the business and the daily life of metropolitan Sydney.

The SHT comprises three main sections as well as the transition structures located between them: twin 940m land tunnels on the north shore, 960m immersed tube (IMT) section made up of eight precast units and twin 400m land tunnels on the south shore (see Fig 1). Concrete durability in the marine environment was given special attention in the design of the SHT structure. Both concretes and structures were required to meet a combination of strict performance and prescriptive properties.

Connell Wagner Pty Ltd, the designer of the tunnel structures together with their sub-consultant, the Australian Centre for Construction Innovation of the University of NSW (ACCI – formerly known as the Building Research Centre), were engaged in providing technical advise on the maintenance of this large vital asset shortly after the completion of construction. A comprehensive maintenance program – the Sydney Harbour Tunnel Asset Protection Strategy – was prepared and has been implemented on an annual basis since the tunnel's completion. The primary objective is been to ensure that a minimum of 100 years of service life can be achieved. An annual maintenance program includes clearly defined tasks such as the routine inspection of key sections, detailed investigations of selected areas or specific items, development of remedial specifications and monitoring of the execution of the remedial work, as well as review or development of new techniques for monitoring and trial remedial works.

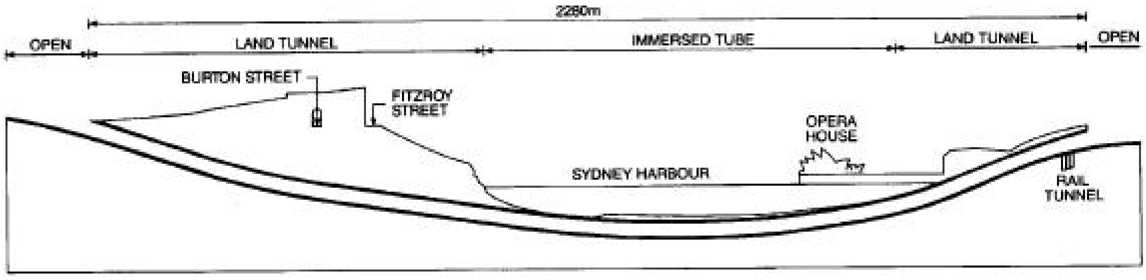


**Fig 1. Longitudinal Section of the Sydney Harbour Tunnel**

The Asset Protection Strategy program has been implemented over the past years. This paper examines some of the techniques used and developed to date in the implementation of this maintenance program.

## 2 SOME TECHNIQUES USED AND DEVELOPED IN THE MAINTENANCE PROGRAM

To ensure the long-term serviceability of any infrastructure, it is important to use the most advanced techniques available, both in the investigation phase and in maintenance. This maximises the likelihood of early detection of defects and may even reduce the remedial work needed. Further an improved understanding of the progress of deterioration and its mechanisms allows appropriate maintenance strategies to be determined and preventive measures to be implemented. Several state-of-the-art techniques have been employed in the SHT Asset Protection Program for on-site monitoring and investigation. These include the GECOR device with a sensor-controlled guard-ring for corrosion rate evaluation, the impact-echo system for non-destructive evaluation of concrete defects and the installation of reference electrodes for monitoring potentials of reinforcing steel near the external surface of the submerged tunnel sections. New techniques and analytical tools have also been developed for more effective monitoring, analysis and remedial treatment. This paper describes some of the main techniques and their applications in the SHT Asset Protection Program.

### 2.1 Crack movement monitoring

Although many modern concretes develop strength more quickly than those produced fifty or sixty years ago, they are more prone to cracking. (Burrows, 1997). Steel reinforcement has been used to control crack width development; but reinforcement does not prevent cracking. Cracks may adversely affect the watertightness of concrete and the need to seal active cracks is common in underground concrete structures. To achieve the effective crack sealing, understanding the nature and the causes of crack movement in a particular structure are essential. Using thin, brittle glass strips as “tell-tales” is a simple practice to detect crack movement. However, based on ACCI’s laboratory testing, a DEMEC strain gauge device was chosen for more effective monitoring of crack movement.

At selected locations, a pair of DEMEC gauge studs was epoxy cemented on either side of a crack. The movement between the gauge studs was then measured regularly with a DEMEC gauge. While theoretically and under ideal conditions the DEMEC gauge can measure a relative movement of about 0.002mm, in practice on a field structure, an accuracy of 0.01mm is more realistic due to the imperfections of the gauge studs, and the operator induced variations.

Over the years it has been observed that crack widths vary seasonally, with cracks becoming wider in winter and narrower in summer. This phenomenon is mostly due to the seasonal differences in the relative humidity and temperature. This finding has lead to the recommendation that to achieve a more effective seal, active cracks should be repaired in winter (at their widest). Furthermore, this technique has also identified that seasonal crack movements in the thinner, unreinforced land tunnel sections are more significant (maximum of 0.2 mm), while the crack movements in the heavily reinforced thick tunnel wall sections are negligible (less than 0.05 mm). These differences in movement have influenced the crack sealing strategy. In the unreinforced land tunnel sections flexible polyurethane resin and foam injection have been used rather than rigid epoxy resin injection. In the thicker, reinforced concrete walls less flexible materials, such as micro-cement grout, can perform effectively. A discussion of the crack sealing materials can be found in the latter part of this paper.

### 2.2 Routine inspection of key areas

Programmed routine inspection of key areas is essential for a facility like the Sydney Harbour Tunnel. Early detection of potential defects can avoid unanticipated risks and may also reduce the cost of remedial work. Such inspections have proven to be invaluable as part of the SHT maintenance strategy.

On the first such inspection, an area with some hollow sounding patches was identified in a cast in situ concrete section of the tunnel. Immediate action was undertaken to secure the area as a precautionary measure. These *drummy patches* were removed with pressure water jetting during the following maintenance shutdown period. A total of approximately 5 square metres was found in this area where cement render had been applied over previous epoxy repair patches. Remedial actions had been undertaken in this area and further monitoring of this area is included in the inspection program.

During another inspection, a small area with a particular crack pattern was found in the roof of a precast section. Further investigation using X-ray Fluorescence Spectrometry and chemical analyses identified that the sample comprised a thin layer of cement slurry. Presumably, this was the excess slurry used to coat the pump line before concrete was pumped into this section and the fine cracks with the particular pattern were probably due to the shrinkage of the cement slurry under the restraint of the surrounding concrete. Remedial measures involved the removal of the drummy surface layer and the repair with an appropriate repair system.

### 2.3 Half-cell potential testing and interpretation

The half-cell potential technique has been used to monitor corrosion in selected areas of this structure from the outset. Half-cell potential testing is a widely used technique for on-site evaluation of the corrosion risk of concrete structures. ASTM C876-91 provides the following guidelines for the interpretation of the half-cell potentials measured with the copper and copper sulfate electrode (CSE) on concrete structures:

- those more negative than -350mV — > 90% probability of active corrosion
- those between -200 mV and -350 mV — 0% probability of active corrosion
- those more positive than -200mV — < 10% probability of active corrosion

However, these guidelines were empirically developed in the United States based primarily on potentials measured in highway bridge decks affected by de-icing salts (Stratfull, 1973, and Van Daveer, 1975). It has since been observed that the characteristic potential ranges can shift significantly for concrete structures under different environmental conditions — such as marine environments or where carbonation of concrete is significant. While Borgard et al. (1991) reported no corrosion for steel at potentials more negative than –350 mV (CSE), Elsener et al. (1990) found severe corrosion at much less negative potentials.

Because the relationship between potential values and corrosion risks differ significantly in different environments, the analysis of risk on the basis of local potential gradients rather than the potential values is recognised to be a better way of identifying active corrosion sites (Elsener and Bohni, 1990). Broomfield (1997) also proposed that active locations in carbonated concrete might be indicated by potential differences greater than 150 mV over a space of one metre.

Based on experience in corrosion investigation in the field, one of the authors, Z. T. Chang has developed a new approach for the analysis of half-cell potential results. Chang firstly noted that locations with a higher potential gradient include not only more negative sites but also more positive ones. However, those locations associated with more negative conditions are characterised by both a higher potential gradient and a more negative potential value. He then developed the *Potential Curvature Method* on the theoretical basis that higher positive curvature values pick up more anodic locations with a higher potential gradient as well as a more negative potential value. A numeric method has been developed to analyse and identify more anodic locations with higher positive potential curvatures.

While further work is still needed to develop the performance criteria linking potential curvature values and the probability of active corrosion, at this stage a characteristic potential curvature value is proposed based on the potential gradient of 200 mV/m. The locations with potential curvatures greater than this characteristic value are considered to have moderate to high corrosion risks of active corrosion. The *Potential Curvature Method* has now been successfully used in many ACCI site investigations of different concrete structures. The method has been shown to be an effective tool for corrosion evaluation in concrete structures, regardless of whether they are in a marine environment or primarily affected by carbonation. Details of the *Potential Curvature Method* are presented in another paper (Chang et al., 2002) in this proceeding.

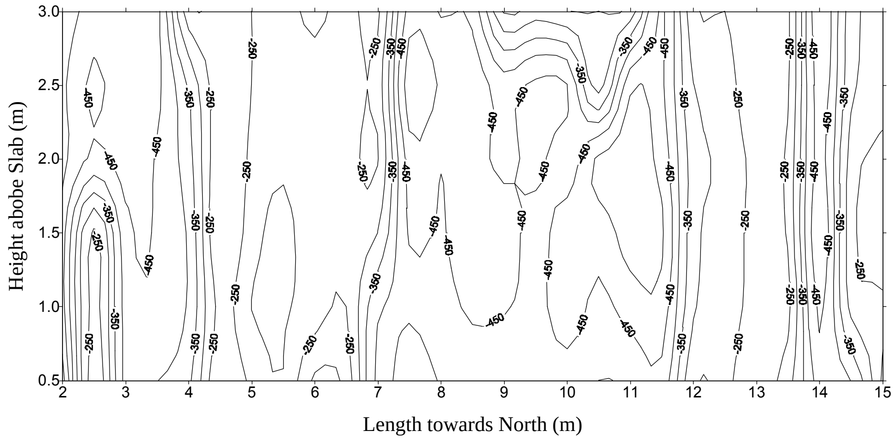


**Fig 2. Half-Cell Potential Contour Map in an Inspected Area**

In one area of the land tunnel sections, half-cell potentials more negative than –350 mV (CSE) had been measured over most of this area. Figure 2 shows the contour map of the half-cell potentials in this area. In spite of these high negative potentials, at two high negative potential locations (one on a crack and the other on uncracked concrete) the steel reinforcement was exposed and found to be free of corrosion.

While a great portion of this area has high negative potentials, the denser potential isolines indicate the locations of higher potential gradients. It is noteworthy however that the denser potential isolines at the x-coordinate of 2.5 m indicate a more cathodic location with less negative potentials. Care must be taken when making assessments based on potential contour maps to differentiate between anodic and cathodic locations.

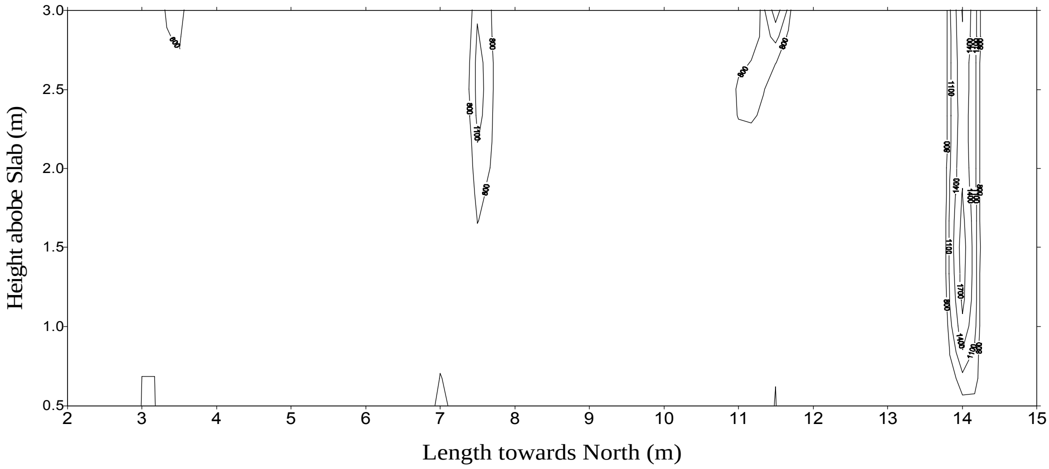


**Fig 3. Potential Curvature Map Highlighting Locations of Higher Corrosion Risks**

Figure 3 shows the potential curvature map of the same area based on the ACCI's *Potential Curvature Method*. Figure 3 shows that despite the high negative potential values measured over this area, the potential curvatures are not significantly high except for the location at the x-coordinate of 14 metres, where there is a crack on a construction joint. A steel bar intersecting with this crack was exposed and exhibited general corrosion over a length of 20mm.

### 2.4 On-site corrosion rate investigation

Half-cell potential measurement and the ACCI's *Potential Curvature Methods* have been successfully used to provide a clear identification of locations with moderate to high corrosion risks. However, the rate of corrosion cannot be determined from these results. For the assessment of service life of existing structures, the rate of reinforcement corrosion at critical sections also needs to be evaluated. Corrosion rate measurements combined with half-cell potentials are also be very useful in the evaluating the effectiveness of remediation measures, such as crack sealing methods and the use of corrosion inhibitors to control the corrosion of steel reinforcement.

The on-site testing of the corrosion rate of reinforcement in concrete is a recent technique. The GECOR device with a "sensor controlled guard ring" is one of the standard pieces of equipment designed for use in the field. Over recent years it has been used in on-site investigations and the following interpretation criteria have been proposed (Broomfield et al, 1994):

| | |
|---|---|
| $I_{corr} < 0.1\ \mu A/cm^2$ | Passive condition |
| $I_{corr} = 0.1$ to $0.5\ \mu A/cm^2$ | Low to moderate corrosion rate |
| $I_{corr} = 0.5$ to $1.0\ \mu A/cm^2$ | Moderate to high corrosion rate |
| $I_{corr} > 1.0\ \mu A/cm^2$ | High corrosion rate |

The corrosion current density of 1 $\mu A/cm^2$ is equivalent to an average corrosion penetration depth of 0.012 mm/year. The corrosion rate measured with the GECOR device is an average value for the rebar surface under the sensor area. However, if corrosion occurs only at a few pits within the area, the average corrosion rate will underestimate the maximum corrosion penetration depth in the pits. Some research has suggested that the maximum penetration of localized pitting could be between 4 to 10 times the general corrosion depth over the steel surface (Gonzalez et al, 1995).

A GECOR device was used to investigate selected areas of the tunnel structure, one of which was the area discussed in the preceding section as having high negative half-cell potentials. The test results showed that the reinforcement at four uncracked locations was all in a "passive" (Broomfield et al, 1994) condition, despite high negative potentials of about –500mV being recorded at two of the four locations. At another two locations intersecting the cracks, the direct GECOR test results indicated one location had a "low to moderate" corrosion rate and the other was indicated as being "passive". However, since the crack zone localises corrosion the direct GECOR test results underestimate the local corrosion rate. A multiplier of 7 was used to estimate the local current densities at locations intersected by cracks. This modification brought the estimated corrosion rate to the category of “low to moderate” at one location and of “moderate to high” at the other. The location with “moderate to high” corrosion rate is on the crack at the x-coordinate of 14 metres where the highest potential curvatures were found with the ACCI potential curvature analysis (see Fig 3).

Generally, the initial results obtained with the GECOR device had fairly good correlations with the locations identified with the potential curvature method and the observation of corrosion conditions of the exposed reinforcement. However, some variables need further investigation. These include the use of an approximate corrosion constant, the assumption that there is a uniform corrosion condition and that the measured steel area is confined under the sensor. These factors need to be studied further both in laboratory investigations and in real structures before the service life of an asset can be predicted based on corrosion rate evaluation.

### 2.5 Development of crack injection technique with micro-cement grouts

Cracks in this structure mostly occurred in the sections that were cast in situ. Most cracks have been repaired using a cementitious material with crack sealing properties derived through crystallisation reactions. Moisture penetration at the repaired locations has been substantially stopped. However, the potential curvature analysis indicates that there is still a relatively higher corrosion risk at some cracks, despite the fact that they have been fully sealed. In order to suppress the corrosion of the reinforcement in crack zones a trial injection of super-fine cement grout was undertaken.

Theoretically, cement grouts are ideal for sealing cracks in concrete. They should suppress corrosion activity by reinstating a high pH environment and by chemical binding free chloride ions. In practice the coarse grain size (up to 100µm) of normal portland cement limits its application for crack grouting. However, the development of micro-cement (MC) products with a maximum grain size smaller than 12µm or even 6µm has created new possibilities.

The first on-site trial was undertaken on two open cracks with a surface crack width of 0.5 mm. The penetration of the MC grout was assessed by taking 90mm long cores on the cracks. Grout was found to penetrate up to 90mm down to a crack width of 0.1 to 0.2mm. One of the two cracks treated is in a reinforced section and the half-cell potentials along the crack were found to shift towards more positive by an average of 77mV twelve months after the injection.

A second recent trial was carried out to inject MC grouts into previously sealed cracks. It was found that the MC grouts could be injected behind surface repaired cracks to 120mm or deeper under a high injection pressure. The half-cell potentials along these cracks are currently being monitored over time to evaluate the effectiveness of MC grout injection on corrosion reduction in the crack zone.

### 2.6 Research on influences of crack width in submerged structures

The influences of cracking and crack width on corrosion of steel reinforcement over the long term are still a topic for debate and further research. Schiessl (1976) investigated the effects of crack width on reinforcement corrosion under marine exposure over a period of 10 years. His test results showed that the effect of crack width (less than 0.5mm) on the steel corrosion rate in the crack zones was insignificant over the long term. Schiessl (1988) explained this to be “Because the cathodic process is the dominating rate determining process, an influence of crack widths on corrosion rate nearly doesn’t exist”. Very few investigations have been conducted on the influences of cracking and crack width on corrosion of steel reinforcement in concrete submerged in seawater. In the UK “Concrete in the Ocean” research program, Wilkins and Lawrence (1983) suggested that transverse cracks up to 0.6mm wide might be tolerable in practice. However, they also warned that, because of the high conductivity of seawater, a galvanic couple might occur with a large cathode (the steel in uncracked concrete) and a small anode (the steel in a crack zone). This could result in significant macrocell corrosion in the crack zone.

The ACCI is currently undertaking research sponsored by the SHT Company into the influences of cracks on corrosion rates of steel reinforcement in concrete submerged in seawater. The details of the test procedures are described in Chang *et al* (2000) and initial test results (Chang *et al*, 2001) have shown that a significant difference was found between the rest potentials of the steel bar in sound concrete and that intersecting a crack when the concrete specimen was immersed in a 3% NaCl solution. A macrocell was formed when the two rebars were electrically connected and, as expected, the rebar intersecting the crack formed the anode.

Further investigations of macrocell current flow between steel sections in uncracked concrete and intersecting a crack have been carried out with the beam specimens as shown in Figure 4. Each beam specimen contains four separate 100mm long rebar sections (S1 to S4 in Fig 4) in uncracked concrete and these rebars are in line with another short rebar (S0 in Fig 4) which intersects a concrete crack. The crack widths in a group of six beams are 0, 0.1, 0.2, 0.4, 0.8 and 3mm. The beams are immersed in a seawater tank and the macro current flow between the rebar sections in each beam is monitored with a Zero-Resistance-Ammeter.

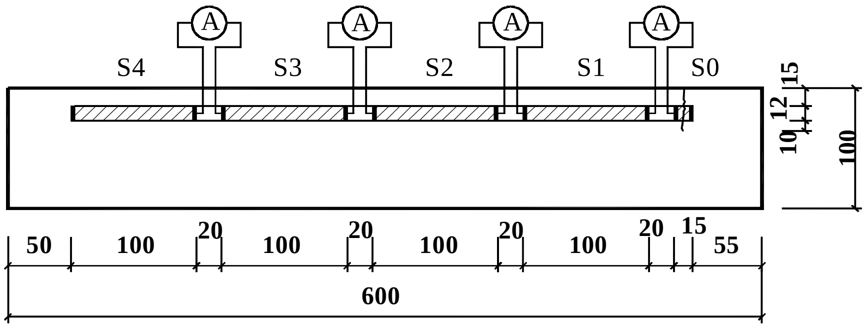


**Fig 4. Concrete Beam Specimens for Macrocell Current Monitoring**

The macrocell current flowing into the rebar section exposed at the crack zone in the cracked beams was initially as high as about 60μA after 7 days immersion, but declined rapidly to less than 5μA after 4 weeks immersion. The macrocell current dropped further to less than 1μA after 24 weeks immersion. This accounted for a low corrosion rate of less than 0.01mm/year for the rebar surface in the crack zone.

A further important finding of this work (Chang et al, 2001) is the electrochemical evidence of the development of a more anodic state in the rebar S1 (see Fig 4) in uncracked concrete next to the rebar S0 intersecting the crack. This was indicated by a shift of the net electric current flow through S1 from the negative to the positive values as shown in Figure 5. The results in Figure 5 were measured from six beams (B1 to B6) with the crack width of 0.1 mm to 3mm. The important implication of this finding is that the corrosion penetration rate of a rebar in the local crack zone could reduce significantly with time due to spreading of the anodic surface and reduction of the macrocell effect.

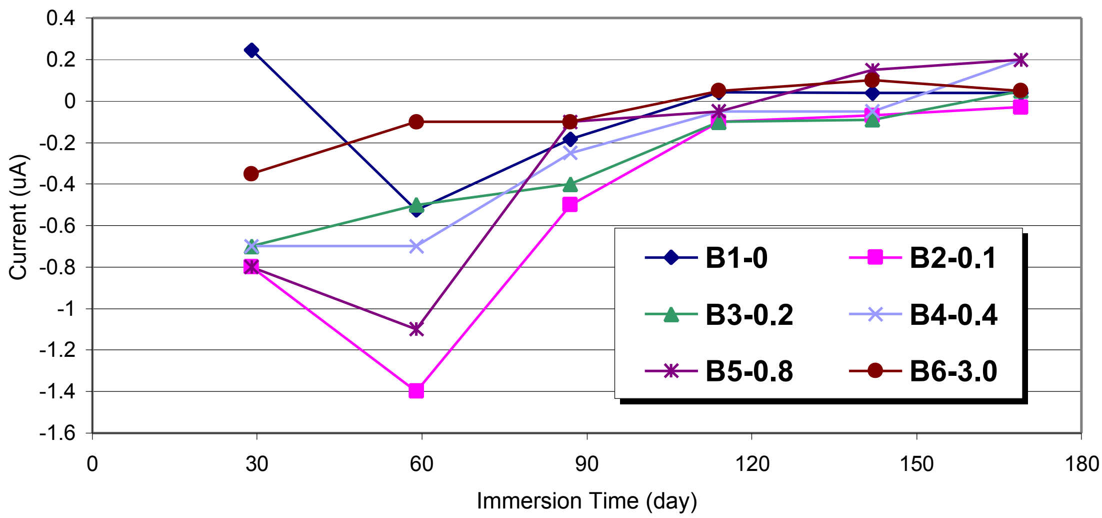


**Fig 5. Net Current Flow through Rebar S1 in Beams B1 to B6 from 4 to 24 Weeks with varying crack widths**

The findings of this investigation also shed light on previous research observations of an insignificant correlation between crack width in concrete and the corrosion rate of steel in the crack zone. This could then be significantly influenced by not only the cathodic process away from the crack zone but also the development of an anodic steel surface adjacent to the crack zone and the subsequent reduction of macrocell effect.

The initial findings of this research work have added to the understanding of the mechanisms of corrosion and corrosion rate of steel reinforcement in submerged marine structures. Such theories are not yet well established. Since on-site investigations of the corrosion condition of reinforcement near the external surface of submerged structures are extremely difficult, laboratory research is needed to investigate the corrosion mechanisms under such special environments. While the current research work is continuing, some on-site investigations are also in progress within the Asset Protection Program. These include monitoring the reference electrodes installed close to the external reinforcing steel and the development of a local cathodic protection system particularly for hot-spot remediation of corrosion activity in the concrete crack zones.

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## 3 SUMMARY

A comprehensive asset protection program has been undertaken for the maintenance of the Sydney Harbour Tunnel since it's opening in 1992. Such a program is essential for the primary objective to ensure that a minimum of 100 years of effective service life can be achieved. State-of-the-art techniques are used in implementing this program. New investigative methods and innovative techniques have also been developed during the course of implementing the asset maintenance. Laboratory research has been undertaken to assist the determination of appropriate maintenance strategies where direct site investigation is restricted. The cooperative work of the consulting and maintenance teams together with the use and development of advanced techniques in an ongoing program has played a vital role in the development of the maintenance strategy of this major asset.

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# Service Life And It's Evaluation Method For Dwellings In The Newly Enforced Building Code In Japan

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**Summary: A "Housing Performance Indication System" based on a newly enforced "Housing Quality Assurance Law" has been implemented since 2000. This system is not compulsory, however, it is expected that high percentages of new dwellings will be applied to this system. Nine performance items including "preventive measures for degradation" which is almost equivalent to durability can be evaluated in the system. Two cabinet orders, "housing performance indicating standard" and "housing performance evaluation standard", were published by the Minister of Construction in order to implement the system. The nine performance items can be evaluated and indicated according to these cabinet orders.**

**This paper outlines the "Housing Performance Indication System" mentioned above and explains how we evaluate and indicate the "preventive measures for degradation" in the system. The indicating system and the evaluation standard for the "preventive measures for degradation" were finally agreed and established after a lot of discussion by experts and consulting with accumulated durability data. This paper also introduces in detail the way of thinking in the establishment of the evaluation standard on the "preventive measures for degradation" especially in the case of steel frame housing as one of the examples.**

**The "factor method" introduced in ISO15686-1 was principally applied in order to constitute the evaluation standard. The performance level of the "preventive measures for degradation" is expressed in a numerical rank out of 3 ranks. The evaluation standard actually consists of deemed-to-satisfy-type prescriptive criteria which are corresponding to each level. However, the concept of the "factor method" is lying in the deemed-to-satisfy-type prescriptive criteria for each rank.**

**Keywords: Factor method, Housing Quality Assurance Law, Prescriptive criteria, Service life, Steel frame housing.**

## 1 INTRODUCTION

A new law, "Housing Quality Assurance Law" was enforced in April 2000 as Gojo (2000) introduced it in the Building Regulatory Workshop held in Canberra. Under this new law, two important measures were implemented by the Government. One is the establishment of the "Housing Performance Indication System" and the other is the enforcement of ten years liability to all sellers and contractors of new dwellings.

This paper firstly outlines the former, "Housing Performance Indication System," then explains "preventive measures for degradation" which is one of performance items in the "Housing Performance Indication System." The paper shows how the "preventive measures for degradation" is evaluated in the performance indication system especially for steel frame housing by way of an example.

## 2 HOUSING PERFORMANCE INDICATION SYSTEM

### 2.1 Background

It has been pointed out that one of the obvious failures found in Japanese housing market is an asymmetry of information held by suppliers and buyers regarding the quality and performance of dwellings. Suppliers of dwellings such as contractors, real estate brokers or house builders generally possess good technical knowledge of dwellings they supply. On the other hand, most consumers possess little technical knowledge of dwellings. Under such circumstances, it is difficult for consumers to make a rational and efficient decision especially in Japan where there are wide varieties of dwellings on the market in terms of construction methods and materials. In other words, consumers need more information with which they can compare the level of quality and performance of one dwelling to another.

In this context, the “Housing Performance Indication System” was implemented. The system is intended to provide consumers with sufficient information concerning the level of performance of dwellings. This information is provided by a reliable specialized body who carries out the evaluation based on the common criteria stipulated by the Minister of Construction. It is important to note that this system is not compulsory but discretionary, therefore, consumers and suppliers have a choice whether to use this system or not. However, it is expected that high percentages of new dwellings will be applied to this system.

### 2.2 Common Criteria Regarding the Level of Performance of Dwellings

To support the “Housing Performance Indication System,” two cabinet orders were published by the Minister of Construction. The one is a “housing performance indicating standard” which stipulates which performance items should be indicated and how to express the level of performance for each performance item. The overview of the “housing performance indicating standard” is shown in Table 1 (Housing Bureau, Ministry of Construction 2000).

The other is a “housing performance evaluation standard” which provides the rules how to evaluate the performance level of dwellings. Contractors and sellers of new dwellings provide a consumer with information on the dwelling they are going to supply, in other words, they enunciates the performance in accordance with the standards under this “Housing Performance Indication System.” According to the standards, the performance level is expressed in numerical ranks for most of the performance items, while for some items, the level is described by quantitative indices or prescription of the design methods.

For instance, the level is expressed as "this dwelling has the rank 1 of energy efficiency out of 4 ranks," or "this dwelling has the rank 2 of earthquake resistance out of 3 ranks". In order to give reliability to the indicated performance level at the time of handover of new dwellings, specialized bodies designated by the Minister of Construction will issue a "housing performance evaluation report" with a special logo. This report is issued after evaluation is carried out in accordance with the standards. The evaluation is conducted twice in the housing production process: the first, at the end of the planning process and the second, during and at the end of the construction process. Correspondingly, the designated evaluation bodies will issue two types of housing performance evaluation reports on the same dwelling.

**Table 1. Performance items in the standards**

| Performance Items | Sub-Items | Identification of Performance Level |
|---|---|---|
| Structural Stability | Earthquake resistance. In terms of the prevention of the collapse of building structures against big earthquakes. | Rank1-3 |
| | Earthquake resistance. In terms of the prevention of the damage of building structures against moderate earthquakes. | Rank1-3 |
| | Resistance against strong wind. In terms of prevention of collapse and damage of building structures | Rank1-2 |
| | Resistance against the snowfall. In terms of prevention of collapse and damage of building structures | Rank1-2 |
| | Load carrying capacity of ground or piles and its investigation method . | -Load carrying capacity<br>-Investigation method |

**Table 1. Performance items in the standards (Contd.)**

| | | |
|---|---|---|
| | Structural method and type of foundation. | -Spread foundation<br>-Structural material and method<br>-Pile<br>- Type - Diameter - Length |
| Fire Safety | Equipment of fire alarms for a fire within a dwelling | Rank1-4 |
| | Equipment of fire alarms for a fire from adjacent buildings | Rank1-4 |
| | Degree of safety for evacuation<br>-Method of smoke exhaustion<br>-Floor planning | -Fire resistance of walls in escape route<br>Rank1-3 |
| | Easiness of escape in case of emergency | a.A balcony connected to the escape stairs<br>b.A balcony connected to a neighboring dwelling<br>c.Escape door leading to the downstairs<br>d.Other measures<br>e.No escape measures |
| | Fire resistance of windows which can catch spreading fire from outside | Rank1-4 |
| | Fire resistance of parts except for windows which can catch spreading fire from outside | Rank1-4 |
| | Fire resistance of party walls and floors between dwellings | Rank1-4 |
| Preventive Measures for Degradation | Measures to extend the life time of main load-bearing components. | Rank1-3 |
| Easiness for the Maintenance | Accessibility for the maintenance of exclusively used piping systems for water supply, drainage and gas supply | Rank1-3 |
| | Accessibility for the maintenance of collectively used piping systems for water supply, drainage and gas supply | Rank1-3 |
| Performance Items | Sub-Items | Identification of Performance Level |
| Thermal Environment | Energy efficiency for heating and cooling | Rank1-4 |
| Indoor Air Quality | Amount of wood-based materials used for the interior finishing (Formaldehyde) | Rank1-3 |
| | Ventilation of whole house | a.Mechanical ventilation<br>b.Natural ventilation<br>c.No measures taken |

**Table 1. Performance items in the standards (Contd.)**

| | | | |
|---|---|---|---|
| | Ventilation of rest rooms, bathrooms and kitchen | | a.Mechanical ventilation<br>b.Windows for ventilation<br>c.No measures taken |
| | Measurement of formaldehyde and some VOCs | | Method, Condition, Results |
| Lighting and Visual Environment | Ratio of opening area in each direction to floor area | | -East : %<br>-South: %<br>-West: %<br>-North % |
| | Ratio of total opening area to floor area for daylighting | | % |
| Acoustical Environment | Degree of sound insulation of floors for heavy-weight impact sound | Measures to upgrade the Insulation | -Upper floor<br>Rank1-5<br>-Lower floor<br>Rank1-5 |
| | | Equivalent thickness of floor slab | -Upper floor<br>Rank1-5<br>-Lower floor<br>Rank1-5 |
| | Degree of sound insulation of floors for light-weight impact sound | Measures to upgrade the Insulation | -Upper floor<br>Rank1-5<br>-Lower floor<br>Rank1-5 |
| | | Reduction level of impact sound | -Upper floor<br>Rank1-5<br>-Lower floor<br>Rank1-5 |
| | Sound transmission loss of party walls | | Rank1-4 |
| | Sound transmission loss of external openings | | Rank1-3 |
| Consideration for The Elderly | Measures for daily safety and comfort of the elderly in exclusively used area | | Rank1-5 |
| | Measures for daily safety and comfort of the elderly in common area | | Rank1-5 |

Under this system a contractor gives the plan evaluation report to the consumer upon entering a contract. The report is to confirm that the contractor is committed to build a dwelling in accordance with the plan, which was evaluated to have the indicated level of performance. Another type of report, which is issued after the construction process, shows that the evaluation body has inspected the construction work and confirmed the accordance with the evaluated plan. The housing performance evaluation report with a special logo has legal force, that is, the contents of the report bind the interested parties once the report is handed from the contractor to the consumer. For consumers, this report not only shows them the performance level of the dwellings clearly but also gives assurance to the results of the evaluation.

# 3 PREVENTIVE MEASURES FOR DEGRADATION

### 3.1 Presuppositions for evaluation of the "preventive measures for degradation"

As shown in Table 1, nine performance items are evaluated in the "Housing Performance Indication System" and most performance items are divided into sub-items. The "preventive measures for degradation" which is almost equivalent to durability is one of the nine performance items of dwellings. This performance item have no sub-items and can be expressed by numerical ranks as shown in Table 2.

**Table 2. Numerical ranks for the "preventive measures for degradation"**

| Rank | Preventive measures for degradation |
|---|---|
| Rank 3 | Necessary preventive measures for degradation are provided for some three generations*. |
| Rank 2 | Necessary preventive measures for degradation are provided for some two generations*. |
| Rank 1 | Necessary preventive measures for degradation are provided to conform to the Building Standard Law. |

*One generation term is supposed to be 25 to 30 years. This term corresponds to average duration for which one generation occupies a dwelling in Japan. Therefore, the rank 3 means the service life of some 75 to 90 years.

The "preventive measures for degradation" is evaluated on the basis of the following conditions.

1) Ranks of the "preventive measures for degradation" express the levels of preventive measures to keep the service life of dwellings by the limit state. The limit state is described as the degraded state when dwellings would reach to any one of states as in below.

   a. The state when the performance or function degrades beyond the allowable threshold, and when it is impossible to recover this degraded state to the allowable limit by means of ordinary repair or partial replacement or renewal.

   b. For the case when the performance or function could be recovered back to the allowable limit by repair, however, the economical disadvantage caused by the successive use of the building in question is expected.

2) The above conditions a) and b) are common ones for all types of dwelling system and the limit state for each structure of dwellings is provided more concretely. For example, the limit state of dwellings for steel frame construction can be defined as the point in time when the cross sectional area of main load bearing components has been decreased by 10% on average due to corrosion. Therefore, deterioration due to other mechanisms such as fatigue, creep behavior, obsolescence, etc. is not taken into consideration in the evaluation.

3) In this evaluation, the service life of main load bearing components are evaluated because it is predominant for the service life of dwellings. Therefore, this "preventive measures for degradation" does not independently describe the service life of non-load bearing components, finishing materials, equipment, etc.

### 3.2 Evaluation practice of the "preventive measures for degradation"

Evaluation of the "preventive measures for degradation" can be performed according to the "housing performance evaluation standard" published as a cabinet order. As to the "preventive measures for degradation", the evaluation standard actually consists of deemed-to-satisfy-type prescriptive criteria which meet each rank. The designated evaluation bodies conduct evaluation of dwellings in accordance with this standard, however, there may be innovative dwellings which can not be evaluated with the evaluation standard. On such occasions, it is possible for suppliers of such dwellings to get approval of the special evaluation method. This approval is conducted by the Minister. Designated testing bodies are accredited by the Minister to do testing or analysis necessary for approval.

The evaluation standard covers dwellings with wooden structures, steel frame structures, reinforced concrete structures, and reinforced masonry structures. For example, outlines of the prescriptive criteria for steel frame dwellings of rank 3 are shown in Table 3 to Table 5.

According to Table 3, for example, the light weight steel frame dwelling whose columns, beams, and braces that are more than 2.3 mm in thickness can meet the requirements of rank 3 of "preventive measures for degradation" if general parts are galvanized with 240 g/m$^2$ on both sides and column base parts of the dwelling are galvanized with same amount of zinc on the general parts and coated with an epoxy primer and an epoxy resin paint on the column bases. (Refer to the underlined part in Table 3.)

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**Table 3. The deemed-to-satisfy type prescriptive criteria for rank 3 steel frame dwellings**

| Thickness of steel | Preventive measures for corrosion* | |
|---|---|---|
| | General parts | Column base parts |
| More than 12 mm | Not necessary | 1) Any coating from Class 2 to Class 5 in Table 4. Or<br>2) Any galvanizing from Class 2 to Class 5 in Table 5. |
| More than 9 mm | 1) Any coating from Class 1 to Class 5 in Table 4. Or<br>2) Any galvanizing from Class 1 to Class 5 in Table 5. | 1) Any coating from Class 3 to Class 5 in Table 4. Or<br>2) Any galvanizing from Class 3 to Class 5 in Table 5. |
| More than 6 mm | 1) Any coating from Class 2 to Class 5 in Table 4. Or<br>2) Any galvanizing from Class 2 to Class 5 in Table 5. | 1) Any coating from Class 4 or Class 5 in Table 4. Or<br>2) Any galvanizing from Class 4 or Class 5 in Table 5. |
| More than 2.3 mm | 1) Any coating from Class 4 or Class 5 in Table 4. Or<br>2) Any galvanizing from Class 4 or Class 5 in Table 5. | 1) Any coating from Class 5 in Table 4. Or<br>2) Any galvanizing from Class 5 in Table 5. Or<br>3) Any galvanizing from Class 4 and coating of "f" or "g" or "h" in Table 5. |

* Columns including base plates, beams, and braces should be protected by the measures shown in Table 3.

The dwelling whose columns, beams and braces are more than 12 mm in thick can also meet to rank 3 of "preventive measures for degradation" if the column base parts of the dwelling are coated with a red-lead anticorrosive paint and a ready mixed synthetic paint. (Refer to the underlined part in Table 3.) Thus, evaluation of "preventive measures for degradation" is performed by consulting with the "housing performance evaluation standard."

There are also additional requirements related to ventilation systems in a roof truss and floor framing in the evaluation of "preventive measures for degradation" for steel frame dwellings, which is not described in this paper.

**Table 4. Coating specifications for steel frame dwellings**

| Class | Specification | Under coat 1 (Number of coating) | Under coat 2 (Number of coating) | Middle coat and top coat (Number of coating) |
|---|---|---|---|---|
| Class 1 | A | Red-lead anticorrosive paint (1) | - | Red-lead anticorrosive paint (1) |
| | B | Zinc rich primer (1) | - | - |
| | C | Epoxy primer (1) | - | - |
| Class 2 | D | High build type zinc rich paint (1) | - | - |
| | E | Red-lead anticorrosive paint (2) | - | Ready mixed synthetic paint (2) |
| | F | Epoxy primer (1) | - | Ready mixed synthetic paint (2) |
| | G | Epoxy primer (1) | - | Epoxy resin paint (1) |
| Class 3 | H | Epoxy primer (1) | - | Epoxy resin paint (2) |
| | I | - | - | Tar epoxy resin paint (3) |
| | J | Zinc rich primer (1) | - | High build type epoxy resin paint (1) |

**Table 4. Coating specifications for steel frame dwellings (Cont.)**

| | | | | |
|---|---|---|---|---|
| Class 4 | K | Zinc rich primer (1) | - | Tar epoxy resin paint (3) |
| | L | Zinc rich primer (1) | Epoxy primer (1) | Epoxy resin paint (1) |
| Class 5 | M | Zinc rich primer (1) | Epoxy primer (1) | Epoxy resin paint (2) |
| | N | Zinc rich primer (1) | High build type epoxy primer (1) | High build type epoxy primer (2) |

*Under coating is performed in a factory. Middle coating and top coating are performed on site.

**Table 5. Galvanizing specifications for steel frame dwellings**

| Class | Galvanizing specification |
|---|---|
| Class 1 | Zinc galvanizing more than 30 g / $m^2$ and less than 60 g / $m^2$ for one side. |
| | Zinc galvanizing more than 60 g / $m^2$ and less than 120 g / $m^2$ for both sides. |
| Class 2 | Zinc galvanizing more than 60 g / $m^2$ and less than 90 g / $m^2$ for one side. |
| | Zinc galvanizing more than 120 g / $m^2$ and less than 180 g / $m^2$ for both sides. |
| Class 3 | Zinc galvanizing more than 90 g / $m^2$ and less than 120 g / $m^2$ for one side. |
| | Zinc galvanizing more than 180 g / $m^2$ and less than 240 g / $m^2$ for both sides. |
| Class 4 | Zinc galvanizing more than 120 g / $m^2$ and less than 180 g / $m^2$ for one side. |
| | Zinc galvanizing more than 240 g / $m^2$ and less than 360 g / $m^2$ for both sides. |
| Class 5 | Zinc galvanizing more than 180 g / $m^2$ for one side. |
| | Zinc galvanizing more than 360 g / $m^2$ for both sides. |

## 4 THE FACTOR METHOD AS THE BACKGROUND OF THE EVALUATION STANDARD

### 4.1 The factor method for predicting service life

As mentioned above, the ranks of the “preventive measures for degradation” can be evaluated by referring to the deemed-to-satisfy type prescriptive criteria as shown in Table 3 to Table 5. However, there may be innovative dwellings with new materials and new preventive measures which are not included in the prescriptive criteria in the evaluation standard. As mentioned, the special evaluation method will be apply to such dwellings. In the special evaluation of the “preventive measures for degradation,” testing and analysis are performed and the service life of such dwellings will be principally evaluated by use of the factor method which was introduced in ISO15686-1 (ISO 2000). It is important to note that the deemed-to-satisfy type prescriptive criteria are actually developed by use of the concept of this factor method.

As Hovde (1998) pointed out, the factor method is one of deterministic approach to predict the service life time of buildings and there are several aspects of the method which need to be further evaluated. However, this factor method is, on the other hand, simple, flexible, and easy to reflect effects of combined degradation factors rather well. This factor method was originally developed as an output of the technical development project of the Ministry of Construction Japan, namely “Development of Techniques for Improving Service Life of Building (1980-1984)” and the AIJ technical committee (AIJ 1993).

Since its development, the concept of the factor method has been actually utilized in the evaluation of service life of limited types of prefabricated dwellings and in developing specification for improving durability of dwellings which can be financed by the Government Housing Loan Corporation. And, this factor method has also utilized for developing the evaluation of “preventive measures for degradation” in the “housing performance evaluation standard.” This is considered to be an epoch in the history of this factor method.

### 4.2 Evaluation of “preventive measures for degradation” by use of the factor method

The way of thinking in the evaluation of “preventive measures for degradation” of steel frame dwellings is explained as follows. Although there are lot of arguments among designers, contractors, suppliers of dwellings and other engineers before obtaining a consensus in the technical committee, the following evaluation concept was finally agreed among them. The service life of steel frame dwellings can be estimated as follows,

$$Y = (Ys + Yz + Yp) \times B \times C \times M \qquad (1)$$

Where, Y : Estimated service life of steel frame dwelling

Ys = Ys0 x Bs : Estimated service life of steel in outdoor condition

Yz = Yzo x Bz : Estimated service life of zinc galvanizing in outdoor condition

Yp = Yp0 x Bp : Estimated service life of coating layers in outdoor condition

Ys0 : Reference service life of steel

Yz0 : Reference service life of zinc galvanizing

Yp0 : Reference service life of coating layers

Bs : Outdoor environment factor for steel in macro level

Bz : Outdoor environment factor for zinc galvanizing in macro level

Bp : Outdoor environment factor for coating layers in macro level

B : Part factor where the components are installed

C : Work execution factor

M : Maintenance factor

As mentioned before, the service life of dwellings are defined as the service life of main load bearing components such as columns, beams and braces in the evaluation of "preventive measures for degradation." It means that the service life of steel frame dwellings ends at the point in time when the cross sectional area of one of such main load bearing components has decreased by 10% on average due to corrosion.

Ys, Yz, and Yp are estimated as follows. Strictly speaking, outdoor environment factors (Bs, Bz, and Bp) should be decided according to macro level location, however, it was difficult to obtain agreed values which were dependent on location of the dwellings and on each materials such as steel, zinc and coating layers. Therefore, a fixed outdoor environment is considered where the steel corrosion rate is 0.05 mm / year and the corrosion rate for zinc galvanizing layers is 11 g / $m^2$ x year and durability of coating layers are fixed as follows,

Ys = ( 0.1 x t ) / ( N x Ks)

Yz = ( 0.9 x Z ) / ( N x Kz )

Yp: Refer to list of estimated service life of various coating systems shown in Table 4. The Service life of Class 1 is considered to be at least 2 years: Class 2 is 5 years: Class 3 is 8 years: Class 4 is 10 years: and Class 5 is 13 years.

Where, t : Thickness of steel

Z : Thickness of zinc galvanizing

N : Exposed surface to corrosive environment (N = 1 for corrosion from one side and N = 2 for corrosion from both sides.)

Ks : Yearly corrosion rate for steel (Ks is fixed at 0.05 mm / year.)

Kz : Yearly corrosion rate for zinc galvanizing (Kz is fixed at 11 g / $m^2$ x year.)

The outdoor environment factors for steel, galvanizing and coating are fixed and it means that the evaluation of "preventive measures for degradation" is performed for only one reference outdoor environment. If the service life of steel frame dwellings in other outdoor environment is needed, it is possible to change the corrosion rate values and service life of coating by investigating the outdoor environment.

The part factor (B) where the components are installed is decided as follows. The part factor values were decided after intensive discussion in the technical committee and consulting with accumulated durability data. Honestly speaking, the durability data are not so enough to support the factor values scientifically, however, the proposed factor values were agreed by most experts in the technical committee as the reasonable values based on their experience and in trial calculations of service life in various conditions.

B = Bk X Bx

Where, Bk : Type of components ( Bk = 0.7 for column bases in the lowest floor and Bk = 1.0 for the other components)

Bx : Environment at micro level where the components are installed. ( Bx = 7.0 for the components exposed to inside and in dry state at all times, and Bx = 1.0 for the components exposed to outside.)

In case of steel frame dwellings, columns, beams, and braces are installed inside of walls. Therefore, the factor B is considered to be 7.0 (1.0 x 7.0) for general components and 4.9 (0.7 x 7.0) for column bases in the lowest floor.

The values of work execution factor (C) and maintenance factor (M) were also discussed among the experts in the technical committee. In the evaluation, a standardized level of work execution and an ordinal level of maintenance are assumed. Namely, steel frame work and other finishing works are carried out, for example, according to JASS (Japanese Architectural Standard Specification) published by Architectural Institute of Japan. Maintenance will be performed in ordinal manners for dwellings. Therefore, the values of the both factors are fixed at 1.0 basically. If the other values are selected for a work execution factor or a maintenance factor, it is necessary to clarify the different points from ordinal work execution or maintenance.

### 4.3 Trial calculation of service life of steel frame dwellings

Here, two calculations of the service life for the two examples mentioned before are shown. As described before, the “preventive measures for degradation” of the dwelling is rank 3 if the load bearing components are more than 2.3 mm in thick and galvanized with 240 g / $m^2$ on both sides and the column base parts are additionally coated with epoxy primer and epoxy resin paint.

The service life time of the dwelling can be estimated as follows,

Y = (Ys + Yz + Yp ) x B x C x M

Where, Ys = ( 0.1 x t ) / ( N x Ks) = (0.1 x 2.3)/(2 x 0.05) = 2.3 years

Yz = ( 0.9 x Z ) / ( N x Kz ) = (0.9 x 240)/(2 x 11) = 9.8 years

Yp = 5 years (Class 2 in Table 4. Only for the column base parts)

B = Bk x Bx = (0.7 or 1.0) x 7 = 0.47 for the column base parts and 7 for the others

C = 1.0

M = 1.0

Therefore,

Y = (2.3 + 9.8) x 7.0 x 1.0 x1.0 = 84.7 years for the general parts

Y = (2.3 + 9.8 + 5) x 4.9 x 1.0 x1.0 = 83.8 years for the column base parts

Then, the “preventive measures for degradation” of the dwelling is considered to be rank 3 in which the service life is ranged 75 to 90 years.

The other example is the dwelling whose columns, beams and braces are more than 12 mm in thick. This dwelling can also meet to rank 3 of “preventive measures for degradation” if the column base parts of the dwelling are coated with a red-lead anticorrosive paint twice and a ready mixed synthetic paint twice. The calculation is as follows,

Y = (Ys + Yz + Yp ) x B x C x M

Where, Ys = ( 0.1 x t ) / ( N x Ks) = (0.1 x 12)/(2 x 0.05) = 12 years

Yz = ( 0.9 x Z ) / ( N x Kz ) = (0.9 x 0)/(2 x 11) = 0 years

Yp = 5 years (Class 2 in Table 4. Only for the column base parts)

B = Bk x Bx = (0.7 or 1.0) x 7 = 0.47 for the column base parts and 7 for the others

C = 1.0

M = 1.0

Therefore,

Y = (12 + 0) x 7.0 x 1.0 x1.0 = 84 years for the general parts

Y = (12 + 0 + 5) x 4.9 x 1.0 x1.0 = 83.3 years for the column base parts

## 5 CONCLUSIONS

The outline of the “Housing Quality Indicating System” which is a part of “Housing Quality Assurance Law” was explained in the paper. This law is considered to be one of the important law which leads to restrain the production of low quality dwellings and through this law more quality dwellings and quality housing products and materials will be supplied in the market.

Among the nine performance of dwellings which are selected in the indicating system, the indicating system and the evaluation standard of the “preventive measures for degradation” were mainly introduced in this paper. The evaluation standard itself consists of a series of deemed-to-satisfy type prescriptive criteia for each rank of the “preventive measures for degradation.”

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However, the factor method introduced in ISO 15686-1 was successfully applied for establishing the series of deemed-to-satisfy type prescriptive criteria together with accumulated durability data.

Since its development, the concept of the factor method has been utilized in limited cases in the actual practice. It is considered to be a epoch that the factor method was successfully applied in order to arrange and to establish the prescriptive criteria of the "preventive measures for degradation."

Of curse, there are several aspects of the factor method which need to be further evaluated. However, this factor method is simple, flexible, and easy to reflect effects of combined degradation factors rather well. As a result, it can be said that this factor method was useful for establishing the prescriptive criteria with the following comments.

1) The "deterministic character" of the factorial method was not inconvenient for developing the prescriptive criteria of "preventive measures for degradation" which related to the durability of the dwellings, because the evaluation standard was established with emphasis not on strict estimation of the service life but on relative comparison in durability of the dwellings.

2) Due to lack of enough durability data, some factor coefficients such as Bs, Bz, Bp, C and M could not be decided for wide range of conditions in the technical committee. Therefore, variation of such factor coefficients could not be obtained and the fixed coefficient (1.0) was selected for such factors. However, it seemed that to fix factor coefficients was not so unreasonable in the evaluation if the effects of such factors were hard to estimate by the accumulated data and experience.

3) In order to establish the prescriptive criteria for each rank of "preventive measures for degradation," a number of trial calculations were done for many cases. It is important to develop scientific deemed-to-satisfy type of prescriptive criteria and it is also important that the developed prescriptive criteria and the evaluation results in the evaluation standard are reasonably corresponding to the experience of experts. A large amount of discussion was also carried out for this purpose.

4) Further improvement of the evaluation standard will be necessary in order to enable evaluation of new types of dwellings, components, and materials. It is also important to obtain feed back data and to review the standards continuously.

## ACKNOWLEDEMENT

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# Maintenance Planning Of Reinforced Concrete Structures: Redesign In A Probabilistic Environment Inspection Update And Derived Decision Making

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**Summary: In the European Brite-Euram research project DURACRETE, a new service life design concept for reinforced concrete structures has been established. This new concept enables the durability design of reinforced concrete structures for a defined lifetime related to limit state formulations. The result of the full probabilistic calculation is a limit state based failure probability of the structure. This new design approach has been used first for a bored tunnel construction in The Netherlands (Gehlen and Schiessl 1999).**

**In this paper the application of this approach is presented for an existing structure, the Olympic Tower in Munich, Germany, mainly in order to facilitate maintenance planning.**

**Taking information from the tender documents into account, the progress of the carbonation front penetrating into the concrete is predicted considering environmental load impact and resistance parameters with their variability. This result is compared with the measured concrete cover. Consequently, a time dependent reliability of the structure against carbonation induced reinforcement corrosion is calculated. The first calculation is updated with inspection data which was measured after 33 years of exposure (carbonation measurements). Based on the updated reliability analysis decisions on future maintenance measures are made.**

**Keywords: Durability design, carbonation, tower, probabilistic approach, reliability.**

## 1 INTRODUCTION

A structure under certain loads may be considered reliable if ultimate and serviceability limit states - both defined by a predetermined probability - are not exceeded by the end of the defined technical lifetime. In the past, structures were designed with regard to durability following a descriptive approach. Consequently, no information about the performance in terms of limit state based reliability is nowadays available. In the past decade much effort has been spend to develop performance based probabilistic design tools. These new concepts can also be applied to existing structures (redesign) in order to receive information of the time dependent performance of the structure.

The television tower in Munich which was build in 1968 is located at the Olympic site and therefore also called Olympic Tower. The overall height of the tower is given as 290.95 m. For construction of the reinforced concrete sections a climbing form has been used. At three levels antenna platforms are located. Above those antenna platforms a restaurant and an additional platform for visitors are located, see Figure 1. The outer diameter of the reinforced concrete shaft varies between 17.86 m at the bottom and 4.5 m at the top.

The tower is exposed to the atmosphere, which is containing usual amounts of $CO_2$ (chlorides are unlikely). The only barrier protecting the reinforcement against carbonation induced corrosion is the concrete cover.

In 1967 the durability design of the reinforced concrete sections was based on a descriptive approach. Consequently, no direct information about the present and future performance of the structure against carbonation induced corrosion of the reinforcement was available.

Within regular inspections of the Olympic Tower some very local areas associated with very low concrete covers with carbonation induced rebar corrosion were detected, mainly at the cantilevers of the antenna platforms. Although no visible signs of corrosion were detected in the structural important areas at the bottom of the shaft (1 m, 8 m and 28 m above ground), the owner questioned the durability.

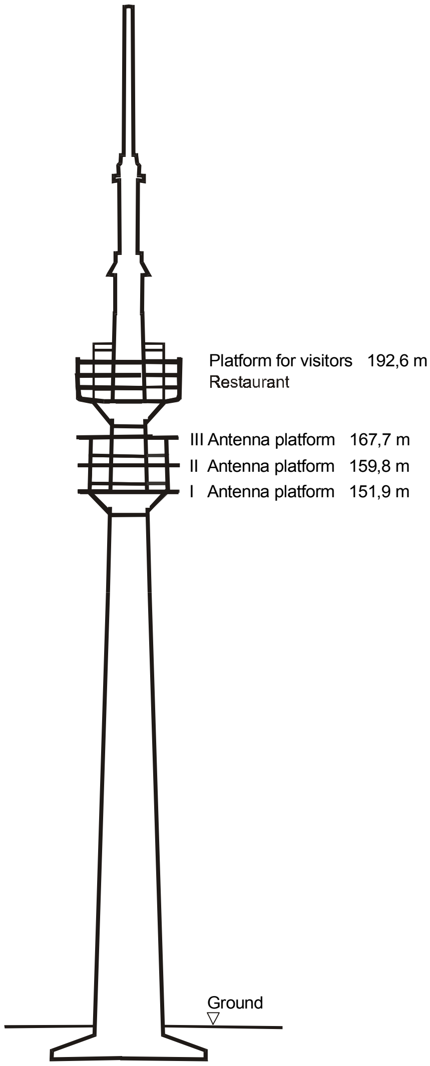


**Figure 1. Olympic Tower, Munich, Germany.**

For further maintenance planning, the condition of the bottom of the shaft was investigated intensively. It was agreed upon to perform a durability calculation of the structure dealing with the deterioration mechanism carbonation induced corrosion. The calculation is based on the given environmental action and the real resistance parameters of the structure. The result of the design gives information about the actual and the time dependent decrease of the structural resistance against carbonation.

For further decisions it was from special importance to get information about the actual carbonation depth, about the expected progress of carbonation, about the concrete cover (non destructive measurement) and finally, taking these information together, to get time dependent information about the probability, that the carbonation front reaches the rebars and corrosion is likely.

Usually, maintenance measures are required if standard requirements in regard to reliability of the structure are not met. According to EuroCode 1 (1991), the maximum failure probability with regard to carbonation induced depassivation (serviceability limit state, SLS) at the end of the service life T of the structure has to be lower than $p_f = p_{target} = 7$ %, equation (1). The targeted failure probability corresponds to a reliability index of $\beta = 1.5$, (EuroCode 1 1991). The so-called reliability index $\beta$ is often used for design purposes (EuroCode 1 1991). The correlation between the failure probability and the reliability index is specified in (EuroCode 1 1991) and illustrated in Figure 2. However, further demonstration of the durability calculations will primarily refer to the definition of failure probability.

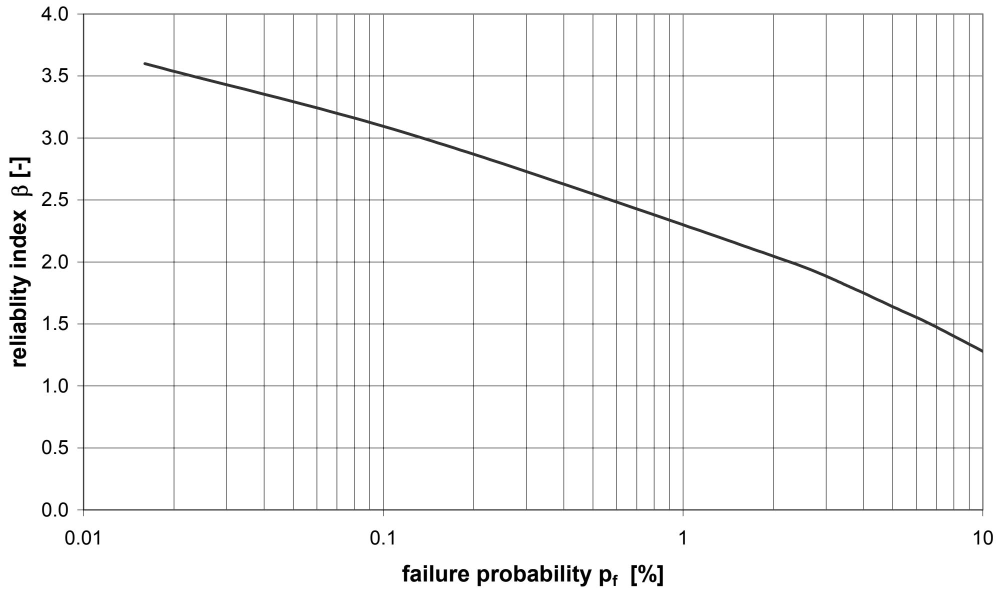


**Figure 2. Interrelation between failure probability $p_f$ and reliability index β for a normally distributed reliability function.**

Based on simplified deterioration models, the probability based durability calculation requires an adequate (deterioration) modelling of the prevailing transport and deterioration mechanism, an appropriate identification of the environmental loading and the stochastic description of the material resistance parameters.

## 2 THE MODELLING, THE DATA

The limit state was set to:

$$p\{failure\} = p_f = p\,\{d_c - x_c(T) < 0\} < p_{target} \tag{1}$$

were the parameters $d_c$ and $x_c(T)$ represent subfunctions:

$$d_c = d_{c,measured} - \Delta d_c \tag{2}$$

$$x_c(T) = \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot \Delta C_S} \cdot \sqrt{T} \cdot \left(\frac{t_0}{t}\right)^w \tag{3a}$$

$d_{c,\ measured}$: non-destructively measured concrete cover

$\Delta d_c$: uncertainty of the non destructive concrete cover measurement

$x_c(T)$: carbonation depth at time T

$k_e$: the influence of environment (e.g. realistic moisture at the concrete surface during use; $RH_{IST}$) on the effective inverted carbonation resistance, $f_e$ and $g_e$ are regression constants

$$k_e = \left(\frac{1 - \left(\frac{RH_{IST}}{100}\right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100}\right)^{f_e}}\right)^{g_e} \tag{3b}$$

$k_c$: parameter taking into account the influence of curing (curing period, $t_c$) on the effective carbonation resistance, $a_c$ and $b_c$ are regression parameters

$$k_c = a_c \cdot t_c^{\,b_c} \tag{3c}$$

$k_t$: parameter which considers the uncertainty of test method [-]

$R_{ACC,0}^{-1}$ inverted effective carbonation resistance of dry concrete including the binding capacity of concrete for $CO_2$, determined at a time $t_0$ at specimens with the accelerated carbonation test ACC in $[(m^2/s)/(kgCO_2/m^3)]$

$\varepsilon_t$: additional errorterm which enables the transfer from laboratory test (ACC) to natural conditions $[(m^2/s)/(kgCO_2/m^3)]$

$\Delta C_S$: $CO_2$-concentration of the ambient air $[kgCO_2/m^3]$

t: time in service [a]

W: parameter which considers the influence of meso climatic conditions (e.g. orientation and corresponding probability of driving rain ($p_{SR}$), time of wetness, ToW), $b_w$ is a regression exponent

$$W = \left(\frac{t_0}{t}\right)^{\frac{(p_{SR} \cdot ToW)^{b_w}}{2}} \tag{3d}$$

$$ToW = \text{days per year with rain } h_{Nd} \geq 2.5 \text{ mm/d} \tag{3e}$$

Statistical information is required to perform a probability-based service live calculation. The quantified stochastic variables for carbonation induced corrosion are given in Table 1. The listed variables in this table have been taken from the equations (2) and (3).

**Table 1. List of stochastic variables influencing the duration of the initiation period (carbonation induced corrosion) Olympic Tower, Munich/Germany**

| *Parameter No.* | *Param. acc. equation **(2)** and **(3)*** | *Source* | *Unit* | *Distribution* | *Mean Value m* | *Standard deviation s* |
|---|---|---|---|---|---|---|
| 1 | $d_c$ | inspection | (mm) | normal distribution | cp. Table 2 | |
| 2 | $\Delta d_c$ | testing equipment | (mm) | rectangular | 0 | 2.0 |
| 3a | $RH_{IST}$ ($k_e$) | weather station | (%) | weibull(max) | m = 76.9 s = 12.2; ω = 100 | |
| 3b | $RH_{ref}$ ($k_e$) | Gehlen 2000 | (%) | constant | 65 | - |
| 3c | $g_e$ ($k_e$) | Gehlen 2000 | (-) | constant | 2.5 | - |
| 3e | $f_e$ ($k_e$) | Gehlen 2000 | (-) | constant | 5.0 | - |
| 4a | $b_c$ ($k_c$) | Gehlen 2000 | (-) | normal distribution | -0.567 | 0.024 |
| 4b | $t_c$ ($k_c$) | tender document | (d) | normal distribution | 1 | - |
| 5 | $k_t$ | Gehlen 2000 | (-) | normal distribution | 1.25 | 0.35 |
| 6 | $R_{ACC,0}^{-1}$ | tender documents in combination with Gehlen 2000 | $(m^2/s/kgCO_2/m^3)$ $[(mm^2/a/kgCO_2/m^3)]$ | normal distribution | $6.8 \cdot 10^{-11}$ (2145) | $3.07 \cdot 10^{-11}$ [969] |
| 7 | $\varepsilon_t$ | Gehlen 2000 | $(m^2/s/kgCO_2/m^3)$ $[(mm^2/a/kgCO_2/m^3)]$ | normal distribution | $1.0 \cdot 10^{-11}$ (315.5) | $0.15 \cdot 10^{-11}$ [48] |
| 8 | $\Delta C_S$ | Gehlen 2000 | $(kgCO_2/m^3)$ | normal distribution | $8.2 \cdot 10^{-4}$ | $1.0 \cdot 10^{-4}$ |
| 9 | T | owner requirement | (a) | constant | 100 | - |
| 10a | ToW (W) | weather station | (-) | constant | 27.3 | - |
| 10b | $b_w$ (W) | Gehlen 2000 | (-) | normal distribution | 0.446 | 0.163 |
| 10c | $p_{SR}$ (W) | weather station | (-) | constant | East: 0.014<br>North: 0.021<br>West: 0.375<br>South: 0.037 | - |
| 10d | $t_0$ (W) | Gehlen 2000 | (a) | constant | 0.0767 | - |

In the following it will be shown how some of the above given quantities of the stochastic variables were obtained. For the other listed parameters further background information can be found in Gehlen (2000).

### 2.1 Parameters 1 and 2, concrete cover dc and measurement error Δdc

The concrete cover has been measured with non-destructive testing equipment. The precision of the measurement depends on the testing equipment. Inaccuracies in measurements are taken into account by the error term $\Delta d_c$ which quantification depends on the testing equipment. Generally this information is provided by the producer of the equipment.

An area of approximately 2 $m^2$ has been measured with a grid pattern each 0.25 m in transverse and 0.25 m in lengthwise direction, each in the levels 1 m, 8 m and 28 m above ground and each in the main geographical orientation of the tower, north, east, south and west. The distribution plot of the measured concrete cover at level 28 m above ground (east direction) is given in Figure 3. The data have been statistically quantified by a computer programme (RCP Consulting 1995). According to the statistical quantification the cover is normally distributed (standard deviation s = 5.7 mm) scattering around the mean value of $d_{c,m}$ = 54.9 mm.

The following Table 2 shows the results of the statistical evaluation for all sets of concrete cover data which have been determined.

**Table 2. Concrete cover data, Olympic Tower, Munich/Germany**

| | | *Result of the Evaluation* | | | | |
|---|---|---|---|---|---|---|
| *Orientation* | *Examination level* | $d_{c,min}$ (mm) | $d_{c,max}$ (mm) | $d_{c,m}$ (mm) | *Standard deviation* (mm) | |
| East, exposition 1 | 1 (1 m above ground) | entrance | | - | - | Normal distribution |
| East, exposition 1 | 2 (8 m above ground) | 37 | 69 | 50.2 | 8.0 | |
| East, exposition 1 | 3 (28 m above ground) | 43 | 79 | 54.9 | 5.7 | |
| North, exposition 2 | 1 (1 m above ground) | 49 | 99 | 72.8 | 17.8 | |
| North, exposition 2 | 2 (8 m above ground) | 35 | 99 | 69.3 | 17.2 | |
| North, exposition 2 | 3 (28 m above ground) | 20 | 79 | 55.2 | 8.7 | |
| West, exposition 3 | 1 (1 m above ground) | 41 | 79 | 57.1 | 8.0 | |
| West, exposition 3 | 2 (8 m above ground) | 50 | 99 | 62.8 | 11.1 | |
| West, exposition 3 | 3 (28 m above ground) | 29 | 99 | 48.2 | 10.2 | |
| South, exposition 4 | 1 (1 m above ground) | 41 | 99 | 71.8 | 17.3 | |
| South, exposition 4 | 2 (8 m above ground) | 14 | 50 | 38.4 | 10.1 | |
| South, exposition 4 | 3 (28 m above ground) | 45 | 65 | 54.9 | 4.3 | |

### 2.2 Parameter 3a, relative humidity, $RH_{IST}$

In order to quantify this parameter the meteorological office closest to the Olympic Tower was dedicated and data concerning the relative humidity of the last ten years were requested. The nearby meteorological office (Munich Town) is in service since 1997. Thus for the years 1991 till 1996 the weather data of the meteorological office Munich-Nymphenburg have been used.

### 2.3 Parameter 6, inverse carbonation resistance, $R_{ACC,0}^{-1}$

The inverse carbonation resistance has been estimated based on Gehlen (2000) taking the few available concrete relevant information into account:

Tower Concrete Mix:

Grade C45/55, CEM I, other relevant information, e. g. concerning w/c ratio, cement content, strength class of the cement were not available.

In order to bridge that gap of knowledge the following already tested composition (Gehlen 2000) was used for the calculation:

Literature Mix:

Grade C35/45, CEM I 42.5 R, c = 320 $kg/m^3$, w/c = 0.50, maximum grain size d = 16 mm, plasticiser FM: 0.5 wt.-%/c. The statistical information of the literature mix was taken from Gehlen (2000).

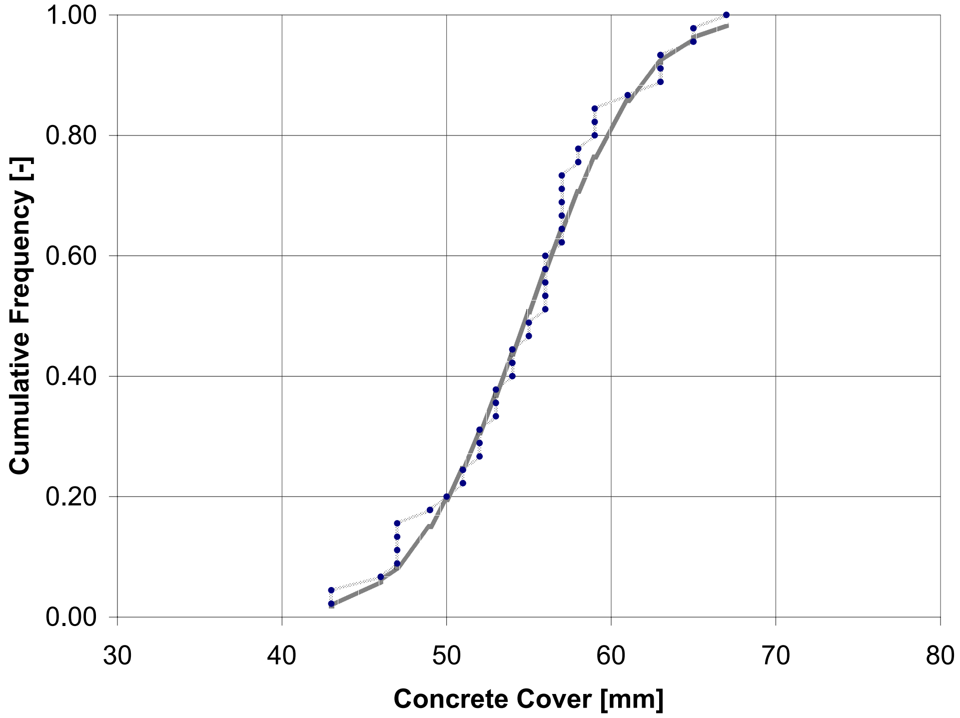


**Figure 3. Statistical evaluation of the concrete cover data, Olympic Tower, level 28 m above ground, orientation: east, Munich/Germany.**

### 2.4 Parameter 10c, probability of surfaces subjected to driving rain, $p_{SR}$

The probability of surfaces subjected to driving rain depends on the orientation of the structure, the geometry and the environmental conditions. This information was also given by the meteorological office.

## 3 THE MEAN VALUE PREDICTION AND THE FIRST FULL PROBABILISTIC CALCULATION

Based on mean values of the quantified parameters it is possible to predict the time dependent process of carbonation (described by equation 3) with an ordinary calculator. The result of such a calculation is given in Figure 4.

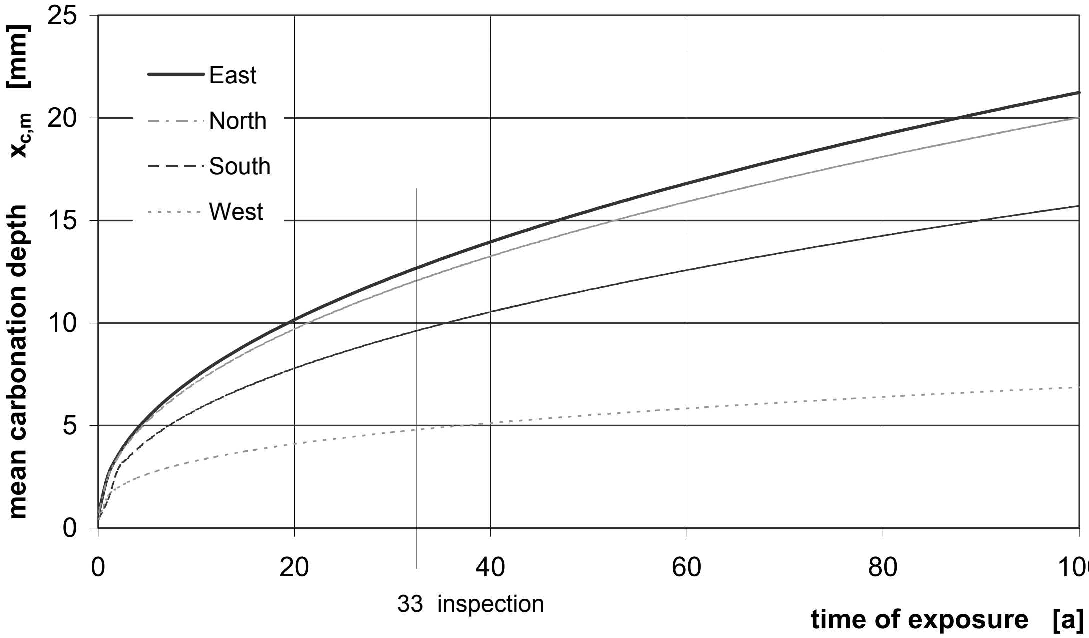


**Figure 4. Predicted mean time dependent progress of carbonation, calculated according Equation 3 with mean values of Table 1.**

The full probabilistic service life calculation has been carried out by a computer programme (RCP Consulting 1995), taking all parameters with their variability into account. The time dependent increase of the predicted failure probability $p_f$, plotted for an exposure period of 100 years is illustrated in Figure 5.

After 100 years of exposure the limit state based failure probability reaches $p_f = 1.4$ %. This failure probability corresponds to a reliability index of $\beta = 2.2$ (Gehlen 2000), Figure 2.

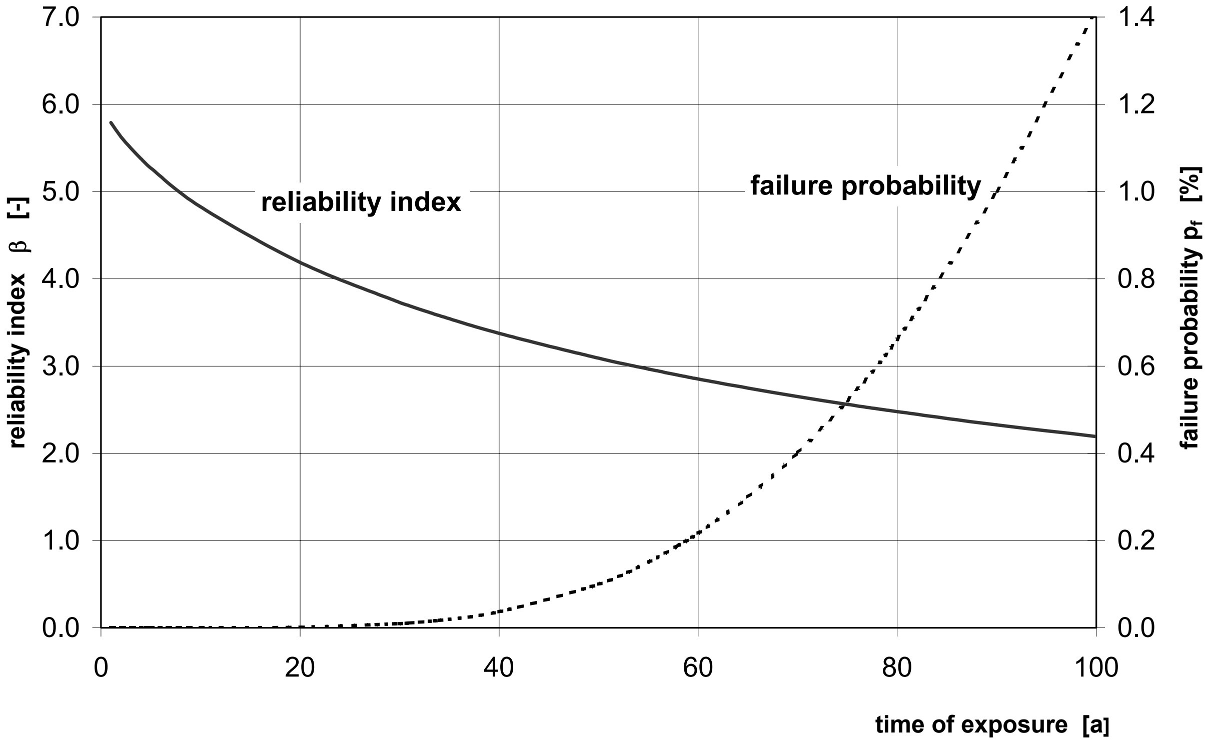


**Figure 5. First probabilistic calculation; time dependent progress of limit state based reliability β and failure probability $p_f$ [%], east direction, level 8 m above ground.**

## 4 COMPARISON OF PREDICTED AND CALCULATED CARBONATION DEPTHS, BASED ON MEAN VALUES

During inspection, the carbonation depth was also measured at all main orientations and all levels (1 m, 8 m and 28 m above ground) of the shaft. In Table 3 the mean values of predicted and measured carbonation depths are compared.

**Table 3. Comparison of the calculated carbonation depth with the recorded carbonation depth of the four expositions, Olympic Tower, Munich/Germany**

| $x_{c,t=33a}$ *in* (mm) *depth of the front carbonation after t = 33a* | *Exposition 1 East* | *Exposition 2 North* | *Exposition 3 West* | *Exposition 4 South* |
|---|---|---|---|---|
| Calculated (cp. **equation 3**) | 12.8 | 12.0 | 4.8 | 9.7 |
| Measured mean value | 10.6 | 3.0 | 2.8 | 4.1 |

One reason for the deviation which has been determined is partly due to the fact that for a lot of parameters a rough estimation (on the safe side) had to be carried out, compare variable $R_{ACC,0}^{-1}$.

The other reason can be found in the circular geometry of the shaft. The progress of the carbonation depends on the moisture of the concrete. In the equation 3 the weather function W(t) has been introduced, which contains the driving rain probability $p_{SR}$ as a parameter. This parameter varies depending to the orientation north, east, south and west, Table 1. The west-east axis represents the main wind direction. The driving rain probability for the east side is low ($p_{SR}$ = 1.4%) which leads to a deeper carbonation front. In contrast the probability of dominant driving rain in the west is larger ($p_{SR}$ = 37.5%), which represses the progress of carbonation. This is confirmed by the measurements. For the west and east orientation the results of the calculated carbonation depth corresponds more or less with the reported one. The larger deviations detected at the north and the south side of the tower were supposedly affected by the circle shaped geometry of the shaft. The explication can be found in the definition of the parameter $p_{SR}$. The probability of driving rain represents the average distribution of the wind direction during rainfall, neglecting the effect of wind-flow around a structure whereby rain is carried with. The normally used probability of driving rain is valid for rectangular buildings, where at least two sides of the building are in the shadow (lee side) of the wind, that means in the shadow of the driving rain, Figure 6, c). At a tower with a circle shaft, like the Olympic Tower, in principle only one side is in the shadow of the driving rain, Figure 6, a), the other sides (windward sides) are exposed to the driving rain. That means, that in principle the Olympic Tower has three west sides (wind respectively driving rain exposed sides) and only one not driving rain exposed side, the east side of the shaft. Under consideration of the real exposition of the south and north side of the shaft, the calculated carbonation depths correspond much better to the reported ones.

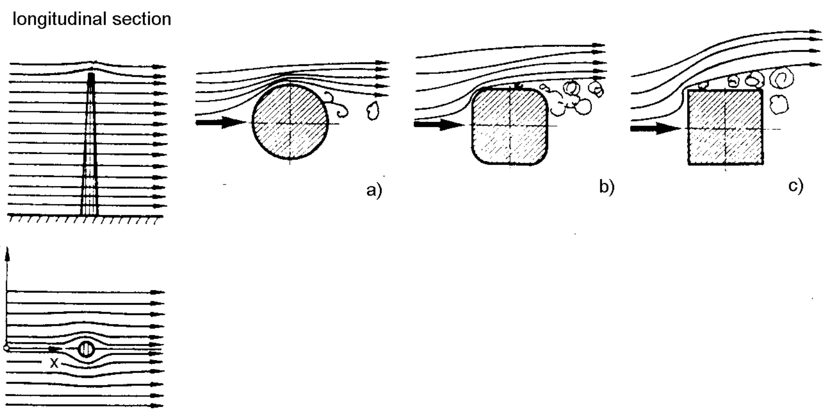


**Figure 6. Wind and driving rain exposed sides of round shafts and rectangular buildings. The figure also shows the vortex trails by Karman, according to Petersen 1988.**

## 5 UPDATING OF THE FIRST FULL PROBABILISTIC CALCULATION THROUGH CONSIDERATION OF THE MEASURED CARBONATION DEPTHS

Through incorporation of the inspection data which have been attained from the structure it is possible to improve the precision of the service life calculation. The update (Bayessches Update) can be accomplished by drafting boundary conditions which take inspection data into consideration. The measurements of the carbonation depth after a service time of t = 33 years represent the inspection data which are required for such an update. The following equation (4) shows the equality constrain which is required to realise the update.

$$x_{c,\,insp.} = \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot \Delta C_S} \cdot \sqrt{\left(t_{insp.}\right)} \cdot W\,(t_{insp.}) \tag{4}$$

$t_{insp.}$: time of inspection (t = 33 a)

$x_{c,\,insp.}$: carbonation depth at time of inspection

In consequence of this additional equation the list of stochastic variables (cp. Table 1) expands with two supplementary parameters. At this point the parameters $x_{c,insp.}$ and $t_{insp.}$ are being introduced whereby $x_{c,insp.}$ represents the measured depth of carbonation at the time of inspection $t_{insp.}$. The following Table 4 (continuation of Table 1), represents the statistically evaluated data which are required for the updated calculation.

**Table 4. Continuation of Table 1, stochastic variables influencing the duration of the initiation period (carbonation induced corrosion), Olympic Tower, Munich/Germany**

| *Parameter No.* | *Param. Continuation of **Table 1** cp. **equation (2)*** | *Unit* | *Distribution Type* | *Mean Value m* | *Standard Deviation s* | *Orientation* |
|---|---|---|---|---|---|---|
| 11-e | $x_{c,inspec}$ | (mm) | normal distribution | 10.6 | 4.4 | East |

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| | | | | | | |
|---|---|---|---|---|---|---|
| 11-n | $x_{c,inspec}$ | (mm) | normal distribution | 3.0 | 1.3 | North |
| 11-w | $x_{c,inspec}$ | (mm) | normal distribution | 2.8 | 1.0 | West |
| 11-s | $x_{c,inspec}$ | (mm) | normal distribution | 4.1 | 1.2 | South |
| 12 | $t_{inspec}$ | (a) | constant | 33 | - | - |

Taking the inspection data into consideration the update of the durability analysis have been accomplished, cp. Figure 6.

The progression of the increase is less distinctive as the one being observed for the first design calculation, cp. Figure 5. After 100 years of exposure the limit state based failure probability reaches $p_f = 0.07$ % (updated design, taking carbonation measurements after 33 years of exposure into account). The calculated failure probability of the update corresponds to a reliability index of $\beta = 3.2$ (Gehlen 2000), cp. Figure 2.

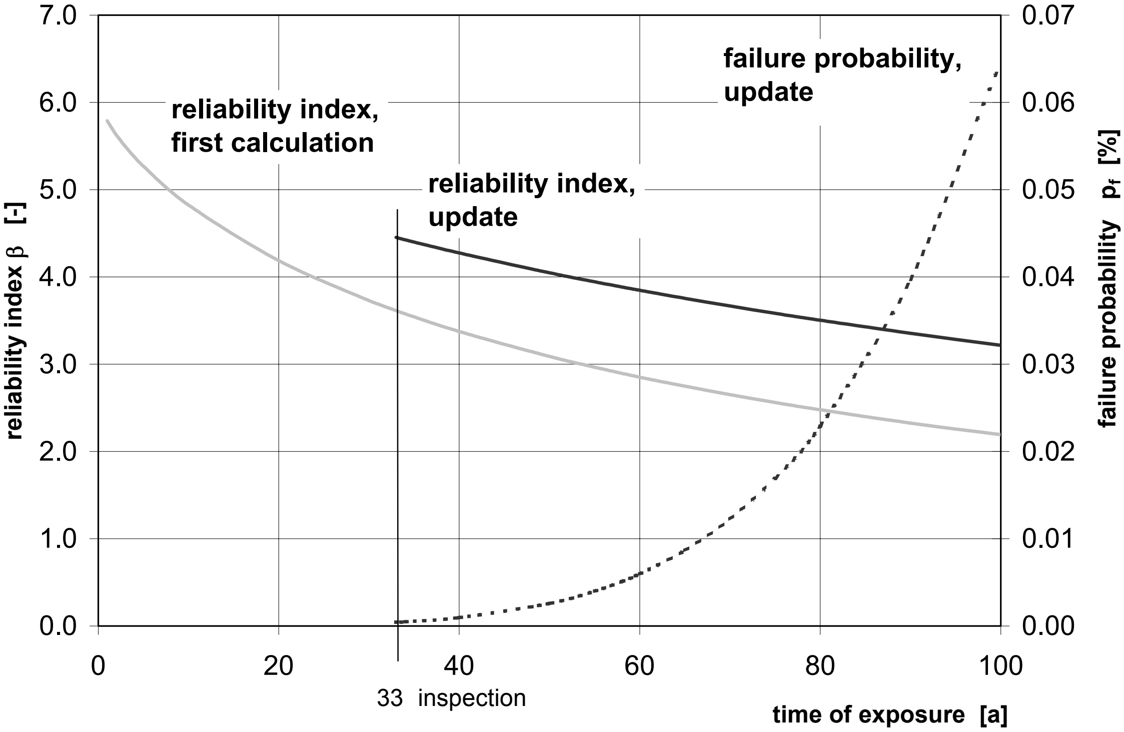


**Figure 7. Update; time dependent progress of limit state based reliability β and failure probability $p_f$ [%,], east direction, level 8 m above ground.**

The deviation between both calculations (first calculation and update) is due to the fact, that not only the uncertain expected mean value of the carbonation depth after 33 years of exposure was detected to be lower but also the variation of the inspection result was lower than predicted one. Both circumstances had an favourable effect on the structural reliability.

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## 6 CONCLUSIONS

A full probabilistic calculation for durability for a existing structure was presented. The investigated results of the reliability analysis which have been introduced can be compared with the requirements of the EuroCode 1.

**Table 5. Indicative values for the target reliability indices b (EC 1, part 1, appendix A, table A2, RCP Consulting 1995)**

| *Limit states* | *Target reliability index $\beta$ (at the end of service life)* |
|---|---|
| Ultimate limit state | 3.8 |
| Fatigue | 1.5 to 3.8 |
| Serviceability limit state (irreversible) | 1.5 |

Compared with the requirements of EuroCode 1, cp. Table 10 and presupposed, the Olympic Tower is classified as an engineering structure with a target service life of 100 years, at least for the investigated surfaces of the shaft an adequate reliability against carbonation induced reinforcement corrosion was calculated,

**Table 6. Calculated reliability indices of investigated expositions of the Olympic Tower in Munich**

| | *Exposition* | $\beta_{target} - t_{100}$ (-) | $\beta - t_{100}$ (-) | $p_f - t_{100}$ (%) |
|---|---|---|---|---|
| 1 | East, level 8 m | 1.5 | 3.2 ✓ | 0.07 |
| 2 | North, level 8 m | 1.5 | 3.7✓ | 0.01 |
| 3 | West, level 28 m | 1.5 | 4.3✓ | 0.001 |
| 4 | South, level 8 m | 1.5 | 3.1✓ | 0.09 |

The outcome of the updated calculation is, that no general maintenance (coating or other protective measures) is necessary.

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# An Integrated GIS Based System For Service Life And Maintenance Planning Of The Building Stock Of Oslo Municipality

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**Summary: Oslo municipality Boligbedriften (BOB) administers 8800 rental objects, and is obliged to do this on a strictly business footing. Therefore it was necessary to implement a new system for documentation of the buildings and the buildings condition, especially the outer parts, including a cost and maintenance planning and optimization module.**

**The new system should also take into account the possibility to document new demands from tenants, owner (the municipality), national and/or international standards/legislations based on more sustainable constructions and the quality of the buildings.**

**The new system, called SLI? S (Service Life Planning and Inspection System) is a further development of the earlier developed Maintenance Management System (MMS) from two EU-projects. It builds heavily on the internationally recognised methodology for prediction of durability, service life and maintenance intervals, now to standardised by the ISO/TC59/SC14 "Design Life of Buildings" in the ISO 15686 series "Service Life Planning".**

**The project runs in 2000-02, and is organised as a joint project between client, supplier/developer and R&D provider. Establishing the BOB building register database (BRDB) is one of the main aims and a core activity of the project. Cornerstones in this work are the Norwegian building register, GAB, and previously conducted research projects on, building condition inventories and assessments, the MOBAK stock-at-risk study, and on establishing dose-response functions, respectively. The GAB register with parcels, properties, buildings and addresses, was established in 1978 and completed in 1995 with digitalized information on almost all 4 million buildings in Norway. Information from GAB is integrated into the MMS CorrCost module. The Norwegian method Ecoprofile for environmental assessment and classification was to be used for recording the environmental conditions.Via those data and building inventories in three steps the BRDB is developed. Other main activities are development of methods, tools and strategies for the total Maintenance plan, further development of MMS into SLI? S, according to the new user needs, and implementation and validation.**

**Implementation of a new system also means system integration with other existing IT systems, as well as changes in the organisation to be able adapt to and make the full benefit of the development.**

**Keywords: Condition and environmental assessment, maintenance Management, Service Life, building register**

## 1 INTRODUCTION

Europe's built environment and infrastructure are rapidly degrading due to environmental impact, lack of resources, methodologies and appropriate data and information systems for maintenance organisation and management. Methods and tools for obtaining a systematic maintenance strategy on the European level are thus major objectives and focus of the EU Action Plans and FW programs.

Two consecutive projects within the EU Framework programmes for Cultural heritage have addressed these needs. The ENV4-CT95-0110 Wood-Assess project, and its successor, the ENV4-CT98-0796 MMWood completed all its tasks and achieved all its objectives in developing a β-version of the Maintenance Management System (MMS) (Haagenrud *et al.*1999 and 2001). The MMS application is a generic software tool to aid the documentation, inspection and maintenance management of cultural buildings. The technology is open and object oriented and can be extended to any kind of building objects.

The theoretical and methodological foundation for MMS builds heavily on the internationally recognised methodology for prediction of durability, service life and maintenance intervals, which is now subject to standardisation by the ISO/TC59/SC14 “Design Life of Buildings” and its ISO 15686 series “Service Life Planning”, which encompasses performance over time functions, degradation mechanisms, damage functions, environmental exposure data, and prediction models (ISO 2000**).** The standards cannot be implemented without the use of ICT tools and extensive data-gathering. As such the MMS concept and application can be directly applied.

The MMS Consortium agreed on the intention and a draft strategy for commercial exploitation of MMS, comprising further development and customizing of the application together with end-users, and building alliances towards providers of Facility Management systems (FM). The present project together with the Oslo Municipality is part of that strategy.

## 2 BRIEF OVERVIEW OF THE MMS

MMS enables the documentation of the building and collection of information regarding its state and condition. It enables the user to integrate and link documents, drawings, and pictures to the building or to any specific part/location/observation of the building, and to link the buildings to maps in a Geographical Information System (GIS), Fig 1.

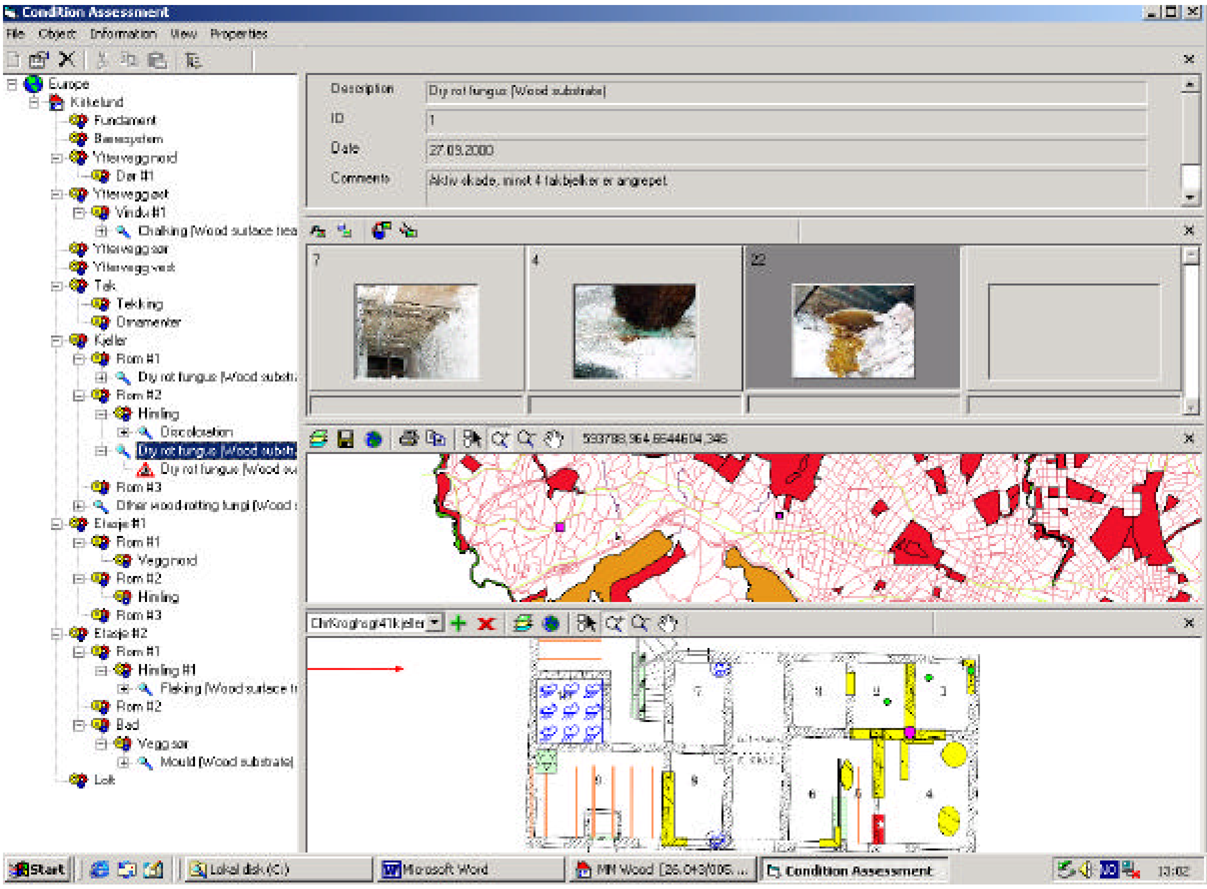


**Figure 1. MMS -Chosen building showing linked information with pictures, maps and drawings.**

The MMS contains several objects, and each object has some related properties. Objects and properties can be defined by the user and put into libraries, and relations between them established. Objects defined here are for example Area/Region, Building Types, Building Part, Inspection Types, Observation and Damage. National standards, guidelines, owners internal systems can be used in creating the libraries, and it can be established in any language (multilingual). The choice of objects and their properties to be assessed can be decided for each installation via the Configuration Menu. The Configuration part makes the MMS very flexible, which is shown by the detailed documentation and assessments performed and reported for the chosen objects. The flexibility puts requirements onto its management and operation, which is best achieved via task definitions for

System Manager and User Manager.Figure 2. MMS -Chosen building showing linked information with pictures, maps and drawings.

MMS has a kernel of four main Sub-Modules: the Condition Assessment -, the Decision -, the Environmental Risk Factor - and the Maintenance Management Module, including also an Inspection Module developed for structuring and performing the inspection work in a systematic and objective way, Fig 2. The Decision Module takes the information from the Condition Assessment Module and the Damage Atlas into consideration when calculating/assessing the seriousness of the damage, and uses the Environmental Exposure module to describe/assess the environmental degradation/risk factors of the site. The Decision Module produces the life expectancy of the objects evaluated. The system structures and reworks information from condition checking and gives specialist a good basis for taking correct decisions. MMS covers the entire work process from documentation gathering, reading in inspection data and photos, analysing and reworking of data, print out of work-card to following up of maintenance work with reports and statistics.

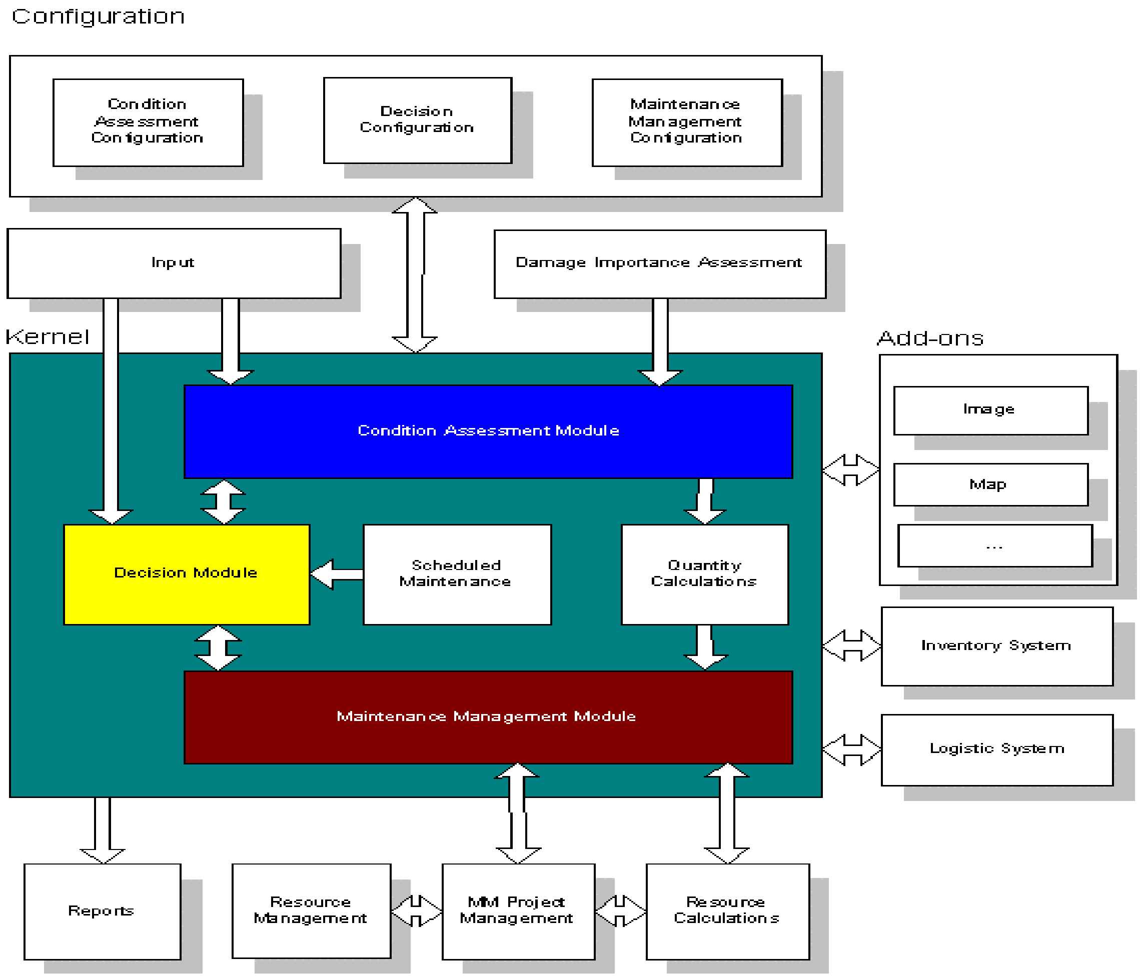


**Figure 3. The main parts of the MMSystem**

The Condition Assessment Module comprises a Condition Assessment Protocol with Damage Atlas, allowing for assessment of damage types and –degrees, causes, effects, consequences, risks, and remedial action for (wooden) buildings. The Damage Atlas contains at present about ten worked examples of various damages for each of the materials/building elements of respectively, Wood constructive, Wood substrate and Wood protected, Adjoining materials in wood constructions, Rendering, Brick and mortar, and Natural stone. The Damage Atlas can be extended with other types of damages on materials and building components. This is one of the issues in the BOB project.

In the Wood-Assess project the methodology for assessing and mapping environmental risk factors and areas for wood on regional, local, and micro scale in Europe was developed. The methodology is developed further adding more materials used in connection with wooden houses, like stone, bricks, rendering, painted rendering, and painted wood, and also making available a module for performing cost-benefit analysis for the built environment (CorrCost).

The ERFM is an important aid in providing necessary data for predicting service life and maintenance intervals for buildings or building elements. The necessary meteorological and pollution exposure data for exploiting the service life functions are collected for the demonstrator regions. The data are either point measurements from measurement stations, or exhibited as simulation models in GIS. By use of the environmental data and service life functions the expected service life are calculated/ modelled for each object/region and material. European standards are used to account for the impact of topography, terrain roughness, sheltering and building envelope form on the local and micro-environmental exposure of the building.

# 3 OBJECTIVES OF OSLO MUNICIPALITY STUDY

The Maintenance Module in MMS was rather general as the EU project criteria just allowed for a couple of objects from each of the countries. A maintenance management plan also has to involve risk assessments and choice of appropriate maintenance strategies, and in order to elaborate that the MMS has to be further loaded and developed with data for specific stock of buildings.

Oslo municipality Boligbedriften (BOB) administers 8800 rental objects (buildings and flats), and is obliged to do this on a strictly business footing, however, also taking into account environmental demands from national policies and regulations. Therefore it was necessarry to implement a new ICT based system for total management of the building stock. As there were no such systems on the market when these demands emerged, BOB decided to take part in the EU project addressing quite a few of the needs (Haagenrud *et al.* 2001).

Following the EU project the implementation project were established between Interconsult as a supplier and BOB as a main demonstration client, with NBI as the R&D provider. The project are being funded by both IC and BOB, and sponsored (25%) by the State Development Fund (SND), and runs in 2000-02. According to their needs the following specific objectives were formulated for BOB

1. To provide a GIS-based total overview and documentation of the existing building stock
2. To develop a system for, and an overview of the condition state, maintenance needs and costs especially for the outer parts, secondly for the inner parts of the buildings
3. Develop and perform a Service Life and Maintenance prediction process on a project and network level.
4. Develop an environmentally sound Maintenance Strategy and -Plan in a 1-3-and 10 year perspective
5. Valuation of building stock and optimisation of leasing income.
6. Total system integration of the MMS with existing and new IT system modules of administration, renting, FM, environmental management, etc

On the part of Interconsult and the MMS consortium the main objectives were to further develop and commercialize the MMS application and services.

# 4 PROJECT STRATEGY AND STATUS

## 4.1 Organisation

The project is organised as a joint project between client, supplier/developer and R&D provider, care being taken to involve relevant users at all the various stages of development and delivery. The sub-projects are

- establishment of the BOB building register database (BRDB) via building inventories in three steps,
- development of methods, tools and strategies for the total Maintenance plan
- further development of MMS into **SLI? S,** according to the new user needs
- implementation and validation

## 4.2 Building register database (BRDB) and building inventories

### 4.2.1 Existing tools and information

Establishing the BOB BRDB is one of the main aims and a core activity of the project. Cornerstones in this work are the Norwegian building register, GAB, and previously conducted research projects on respectively, building condition inventories and assessments, the MOBAK stock-at-risk study (Kucera *et al.* 1993), and on establishing dose-response functions, the UN ECE ICP (Kucera *et al.* 1998). The MOBAK study established average amounts of materials for generic buildings types (offices, single and multi family dwellings, etc) and their degradation depending on environmental parameters, the so – called damage functions.

The GAB register with parcels, properties, buildings and addresses, was established in 1978 and completed in 1995 with digitalized information on almost all 4 million buildings in Norway. Information from GAB is integrated into the CorrCost module, which is also a module in the MMS system. On contract for the Air Pollution Authorities a total cost-benefit analysis was conducted in 1996 for Oslo using this module, and a set of damage functions and stock-at risk data from the UN ECE and the MOBAK studies, respectively (CIB 2000, Glomsrød *et al.* 1996). In CorrCost the study area is divided into grids, the finer the grid resolution the more accurate the calculations. It allows counting of different types of buildings and building materials with different economic value to be done for each grid. Maps have been produced showing exposure environment and the stock of materials and buildings at risk by integrating an information layer on the building register GAB.

Fig. 3 shows the location and number of different types of buildings in grid no. 20-22 (500x500m), together with the environmental characteristics of that grid (Haagenrud and Henriksen, 1996). By use of the materials distribution factors and the environmental dependent service life and maintenance intervals from the MOBAK study the costs can be assessed for each material and building type within each grid. Such calculations have been performed for 1586 grids of the greater Oslo area.

In 1998 about 1/3 of the stock of buildings belonging to BOB was extracted from the Oslo CorrCost database and subjected to a similar study (Haagenrud *et al.* 1998). Based on the MOBAK key figures, types, amounts, and distribution of materials as well as maintenance costs were calculated. The maintenance costs for the building fabric, were estimated to about 25% of the total costs for operation and maintenance, and of which about 15% was due to air pollution. Compared to key figures for total building maintenance costs the assessments and calculation showed the maintenance costs for the outer parts to be about 25%, which were considered reasonable.

Parallell to the technical condition documentation and assessment, an environmental assessment of the buildings was also to be performed as basis for developing BOB’s environmental strategy. The Norwegian method Ecoprofile for environmental assessment and classification was to be used for recording the environmental conditions (Pettersen *et al.* 2000)

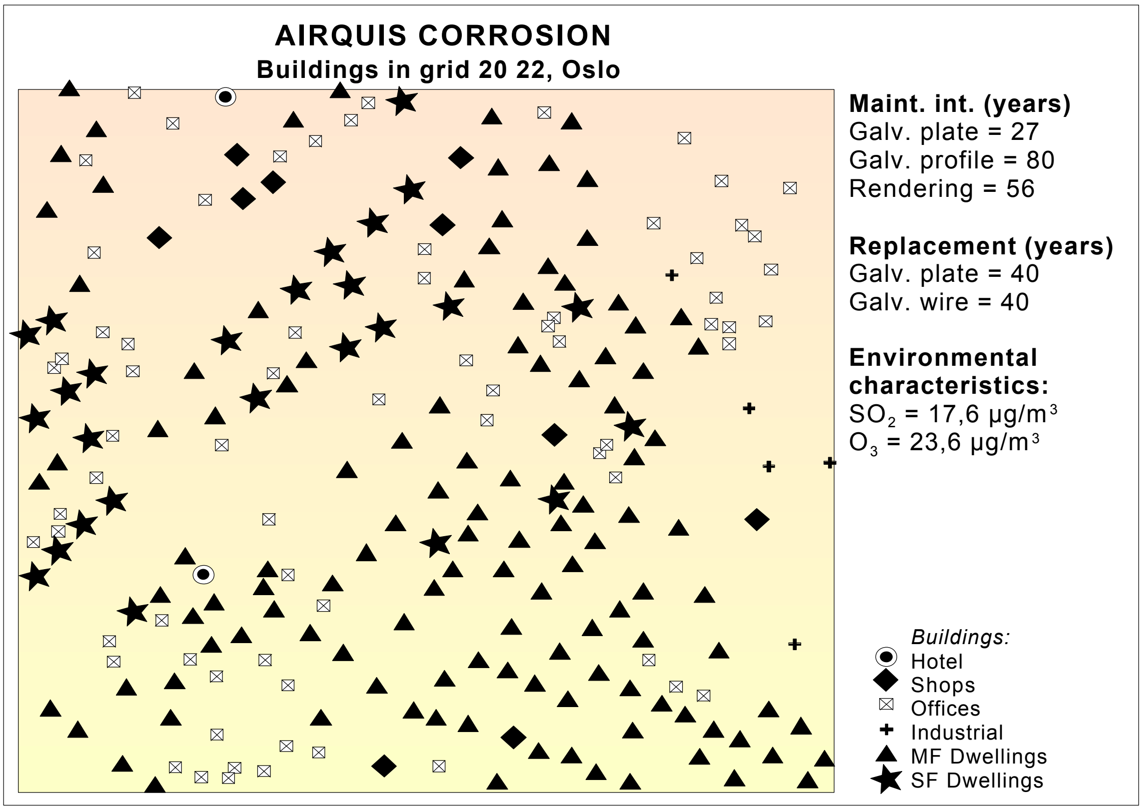


**Figure 4. Characteristics of buildings and exposure environment in one of 1584 grids (500x500 m) in Oslo, exhibited in CorrCost (from Haagenrud and Henriksen, 1996).**

4.2.2 Present project status

In the present work the building of the BRDB involves completion of the register with the MOBAK type of data, redefinition of building information elements, and thereafter replacement of a sufficient amount of the MOBAK type of data with real data from field inspections.

The real data are both basic and historical data about each building object. This implies

- calculation of real amounts of materials and components, which is done by automatic calculation from real drawings that are digitalized and put into the MMS application
- evaluation, adjusting and complementing the GAB register
- data gathering by field inspection in three steps. The first preliminary phase (Phase 0) were 3-5 buildings for educational and processual purposes. Phase 1 included about 25 buildings and 1000 flats, with the main aims to develop the main information elements structure and the accompanying formats to be used for registration, to organise and train the data gathering inspection teams (at least two inspectors from each of the three partners), and then to carry out the inspections. All

inspections in this phase are on Level 1, i.e. visual inspection.The final Phase 2 will the final testing of the MMS tools and procedures, and completion of the BRDB to allow for maintenance planning at chosen level of significance, and for fulfilling the other objectives defined.

The Phase 1 field study is just completed, and data analysis are going on. About 20 different forms have been developed for the inspection, allowing for in principle all components and main functionalities to be assessed. The forms follow the outline of the condition assessment protocol, as exhibited in the Damage Atlas (Haagenrud *et al.* 2001). These forms will be evaluated and eventually amended before they are imported into the MMS isnpection module, i.e they will be made available on PDAs. The next phase is scheduled for spring 2002.

4.2.3 Environmental classification with Ecoprofile

The Ecoprofile classifications were performed simultaneously with the technical inspections on all buildings. Figure 4 shows the results from one of the buildings.

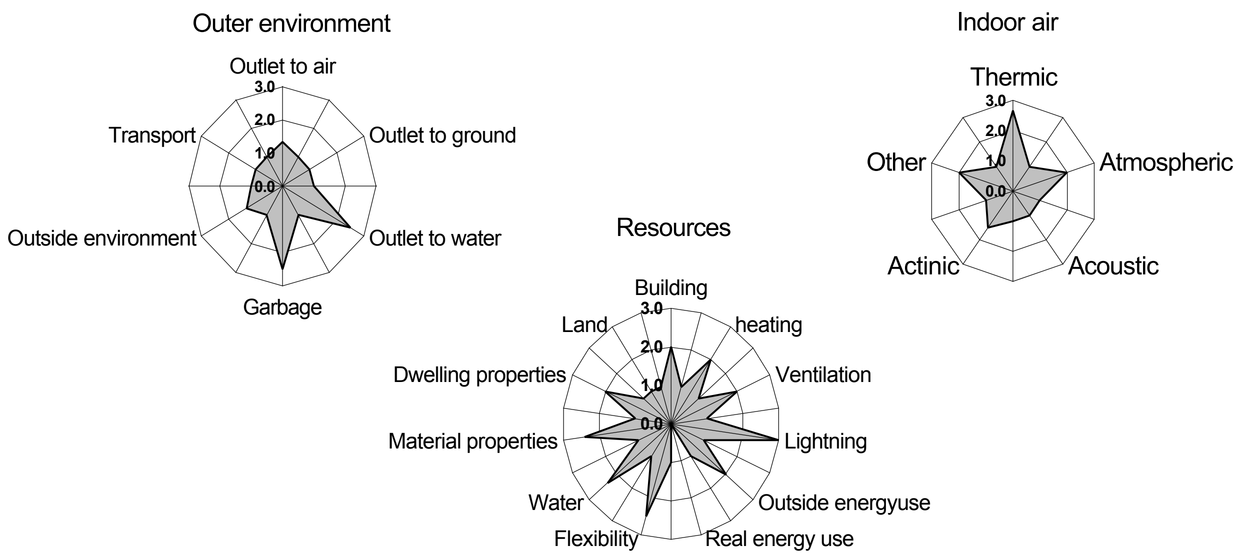


**Figure 5. Radar diagrams showing the environmental classification for External environment, Resources and Indoor climate.**

The Ecoprofile of a building is divided into the three principal components ”External environment”, ”Resources” and ”Indoor Climate”. These components are then divided into sub-components with different consequences for the principal components, and therefore weighted. Several of the sub-components also have underlying sub-components. Each sub-component and underlying sub-component contains a number of parameters that are individually evaluated and classified according to a scale from 1 to 3 where:

- Class 1 = Lesser environmental impact
- Class 2 = Medium environmental impact
- Class 3 = Larger environmental impact

The parameter classification adds up and are averaged and weighted into a similar grade for each of the sub-areas, as shown.

## 4.3 Development of Maintenance Strategies and Plans

One of the main project objectives is to develop a protocol for service life optimisation of the maintenance management for the building stock. This includes development of methodologies as well as tools and techniques. In completing Phase 1 of the BRDB, a basis for this development has been created and work is going on.

The MMS-concept is both a software system and an organising framework to help the building stock managers in the process from inspections and registration of the buildings stock to service life performance analysis and optimisation. Traditionally maintenance analyses are carried out by single project analysis of single building that is merged together into a periodic maintenance plan with a bottom-top approach as shown in Figure 6. This approach does neither take into consideration the life cycle cost and -performance of the separate building or the building stock as a whole.

In the Oslo municipality study the maintenance analysis and planning, which has a top bottom approach as shown in Fig. 5, will be performed on two levels:

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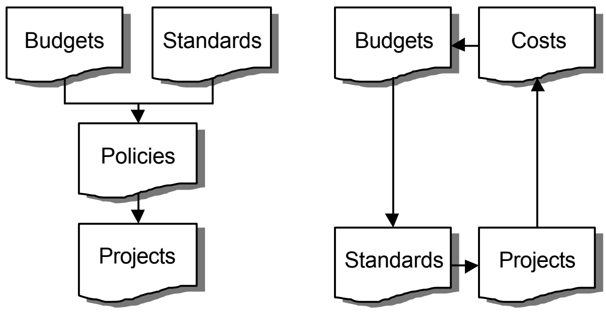
- building stock level, where the objective is to develop long term maintenance strategies for the building stock as a whole including funding needs
- building level, where the objective is to develop both short and long term maintenance plans and programs including budgets

The results from the analysis of the building stock are forming the basis for proposals for budgets, including fund needs and forecasted future consequences of various funding scenarios, as well as strategic policies and plans including levels of budgets.




*TOP DOWN APPROACH:*

*Budgets and standards are used to develop optimal policies which are then used to plan projects. Feedback is provided to refine the models. Budgets and standards may be modified to perform what-if analyses*

*BOTTOM UP APPROACH:*

*Standards assist in planning projects that are totalled to generate costs which are then compared to budgets. This is used to adjust the standards and modify plan.*

**Figure 6. Maintenance management philosophies.**

At building-level, the objective is to determine the best strategy for a selected building or facility within imposed constraints including available funds. The primary results of project-level management include an assessment of the cause, extent and distribution of degradation, identification of possible maintenance actions and strategies, and selection of the "best" strategy given the constraints present.

For the long-term strategic analysis at building stock-level several approaches to decision analysis will be investigated, with a primary focus on Markov chain models (Golabi *et al*. 1982, Carnahan *et al*. 1987, Lounis *et al*. 1998). In this methodology, facility condition is represented by a discrete state, and the deterioration process is modelled as a Markov process. By using the CorrCost and MOBAK models together with the results from the condition surveys when establishing the transition matrix, both the properties of the building stock and the environmental exposure conditions can be taken into consideration in the analyses.

At the building level separate maintenance plans and programs will be developed based on service life analysis of buildings and building components, taking both pure monetary cost and environmental impacts into consideration.

### 4.4 Development of MMS into SLI? S (Service Life Planning and Inspection System)

The further development and customizing of the MMS into the new **SLI? S** follows the obligatory and standardised model for developing such products, namely User requirements, functional specifications, application building and validation.

Important requirements for the new system are that information should only be updated once, the same information should be used by different modules in the system, and that the updated information should be available for persons with different roles in the organisation. Another essential requirement is that the new **SLI? S** should be fully integrated with the existing Facilty Management system.

According to plans the validation of **SLI? S** will start Spring 2002.

## 5 CONCLUSIONS

Oslo municipality Boligbedriften (BOB) administers 8800 rental objects (buildings and flats), and is obliged to do this on a strictly business footing, however, also taking into account environmental demands from national policies and regulations. Therefore it was necessarry to implement a new ICT based system for sustainable management of the building stock.

The new system, called **SLI? S** (Service Life Planning and Inspection System) is a further development of the earlier developed Maintenance Management System (MMS) from two EU- projects. It builds heavily on the internationally recognised methodology for prediction of durability, service life and maintenance intervals, the ISO series “Service Life Planning”. The project runs in 2000-02, and is organised as a joint project between client, supplier/developer and R&D provider. Main activities are establishment of the BOB building register database (BRDB) via building inventories in three steps including environmental assessments with Ecoprofile, development of methods, tools and strategies for the total Maintenance plan, further development of MMS into **SLI? S** according to the new user needs, and implementation and validation. Implementation of a new system also means system integration with other existing IT systems, as well as changes in the organisation to be able adapt to and make the full benefit of the development.

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## 6 ACKNOWLEDGMENTS

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# Integrated Maintenance Management Of Facilities

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**Summary: The performance of multi-systems' buildings depends to a high degree upon the efficiency of the implementation of the maintenance strategy. Owners of such facilities require that their maintenance departments demonstrate efficient management and performance of built-assets. However, management-reporting procedures provide a little in the way meaningful measures of built-asset performance or efficiency of use of resources for maintenance. The research is aimed at development of quantitative indices for appraisal of maintenance activities in multi-systems' built-assets. The research is focused on hospital buildings in Israel as a model for multi-systems' facilities. The method of research includes a comprehensive structured field survey that included an interview with the chief-engineers of 17 hospital facilities followed by a survey of building performance in these facilities. Four principal indices were developed for the evaluation of maintenance: (1) Building Performance Index (BPI)- expressing the efficacy of maintenance as seen in the state of the building and of the different systems in it; (2) Manpower Sources Diagram (MSD) – expressing the ratio between in-house and outsourcing of manpower for maintenance; (3) Maintenance Input Index (MII) - a quantitative tool for determining if the maintenance inputs indicate their efficient use relative to the physical and performance state of built-assets. The Managerial Span of Control (MSC) – This index reflects the number of subordinates per manager in the maintenance department. The model was implemented in a comparative case study on a couple of hospitals. The research concludes that integrated implementation of quantitative indices is essential for the effective maintenance management of multi-systems' facilities.**

**Keywords: Maintenance Management, Built Environment, Sustainability, Life Cycle Costs, Value Engineering**

## 1 INTRODUCTION

The maintenance of a building is defined as "The sum total of actions taken in the building with a view to preserving its components in good and serviceable condition" (Israel Standards Institute, 1996). In recent years there developed a tendency towards the erection of Intelligent buildings, to the extent that, today, every new building being constructed includes advanced sophisticated systems, both in response to functional demands and for the convenience of its users.

The performance of multi-systems' building depends to a high degree on continual and planned periodical maintenance. This is far from simple, since the building may simultaneously serve a great many areas of endeavor and activities: office floors for work during the day, floors for commerce and services, areas for recreational activities in the evening, and so on. Accordingly managing the maintenance of a multipurpose building demands careful and precise planning based on a well-organized maintenance program. Furthermore, decision makers concerned with the maintenance of multi-systems' buildings are not infrequently called upon to decide whether they should have works in the various disciplines of maintenance carried out by a permanent internal work force (in-house provision) or by an external contractor (outsourcing). The present research focused on hospital buildings in Israel, which are characterized by a multitude of mechanical and electrical service systems and by high demands of serviceability and thus constitute a model for the maintenance of multi-system buildings. This paper presents a quantitative approach to decision-making in the maintenance of multi-systems buildings. The approach referred to is based on indices for assessing the performance of the buildings, the costs of maintenance and of manpower, and the efficiency of the use of resources.

## 2 LITERATURE BACKGROUND

The past three decades (beginning with the seventies) have seen a change in the character of our conception of the management of maintenance works. On the one hand it was the total-system view that the person engaged in the management of building construction as a whole and of the management of maintenance in particular had to apply in the execution of his task. On the other hand the background and the understanding required of him in all the domains of the building, have brought it about that the discipline of building engineering now includes not a few elements of the theories taught in the behavioral sciences, mainly because of the "multi-faceted management" it implies. (Quah, 1992).

The Israel Standard defines the need for carrying out maintenance, stating that "Maintenance operations in a building are intended to preserve the proper performance of the building, its outward appearance, its economic value, and so, too, the health and the safety of its users, of its visitors, and of those that pass by it." The standard comprehensively reviews the different systems of the building, citing examples of possible faults in them. In addition the standard also points to the activities that are likely to constitute a risk to the good repair of the different elements of the building. To quote but one example, on the subject of the foundations (in the chapter dealing with the skeleton) the standard presents a list of possible shortcomings of these elements, such as cracks, fractures, weakening of the concrete, and exposure of the reinforcement. It also contains a list of activities and occurrences that represent dangers to the serviceability of a particular element (Israel Standards Institute, 1996). The gist of the discussion is presented in the form of a table in which, against each component, are noted the required maintenance activities, the frequency of their execution, and the person responsible for their performance. That discussion is held in the chapters dealing with the sanitary, electrical, air-conditioning, elevators, and fire precaution systems. The Israeli standard leaves the user no discretion but obliges him to act in accordance with certain injunctions laid down and pointed out in it. Non-execution of these injunctions is liable to cause the building, or its systems, to suffer failure or to render it incapable of fulfilling the demands of serviceability.

Homer, El-Haram, & Munns (1997), in their research, stressed that the maintenance of a building has many purposes, of which the most prominent are:

- -To ensure that the building and all its systems are fit and safe to use.
- To preserve the value of the components of the building.

In their research the investigators try to map the different approaches common in maintenance policy, subdividing them into three categories: Breakdown, Preventive, and Condition-Based Maintenance:

<u>Breakdown Maintenance</u>: This is the simplest strategy for carrying out maintenance work. Under it every element is used until it fails or no longer succeeds in properly fulfilling its task. This type of maintenance does not need any planning ahead of maintenance operations. Accordingly, application of this strategy is recommended when the failure of a given component does not lead to high repair costs and also when its day-to-day performance level is not high.

<u>Preventive Maintenance</u>: This strategy is based on the goal to minimize the chances of a sudden failure of a component. The principal point of the method is constant periodical treatment at known time intervals of each of the building's components. The advantages of preventive over breakdown maintenance are as follows:

**a.** Since maintenance is pre-planned, it can be carried out in the manner most convenient for the users.

**b.** It assures conservation of the good repair and completeness of the building at all times, practically without its operating under conditions of failure.

**c**. Reduction of the cost of maintenance by preventing "concatenations of mishaps".

**d.** Improvement of the convenience and the safety of the users of the building.

**e.** Savings in the cost of works as a result of the ability to control the time of execution.

On the other hand there are certain disadvantages to the approach, namely:

**a.** Many components are replaced when they are still able to serve the users' purposes for further periods of time (the replacement of components does not necessarily conform to their actual service life). The result of this approach is over-maintenance.

**b.** Preventive maintenance requires available manpower in full employment.

<u>Condition-Based Maintenance</u>: This approach is defined as "maintenance carried out in response to a significant deterioration in the condition, or the performance, of a given component of the building." The intention is not only to carry out a wide-ranging (and pre-planned) examination, but also constantly to arrive at decisions with respect to each component, if it is, or is not, capable of continuing to fulfill its purpose. This approach was developed for dealing with buildings the cost of failure of which is considerable, but where the cost of replacing components on the basis of fixed timetables is liable to be superfluous or may even be the cause of unnecessary failure (power stations, manufacturing plants, and the like). This particular approach is more economical and contributes to the overall profitability of the object. Condition-based maintenance deals with the causes of failures or mishaps, not on their symptoms (Barnard, 1996).

Neely & Neathammer (1991) examined the maintenance of buildings on four army camps situated in various parts of the U.S.A. with a view to developing a model of the costs of maintenance of each type of building. The buildings were subdivided into 34 categories according to construction, use, and the like. In each of the 34 categories the maintenance expenses over the 120 years of service life of a building were examined. The analysis of the findings for hospital buildings shows that the lion's share of the maintenance work cost – close to 32% - was spent on the various items of interior finish. Second place was occupied by the air-conditioning system, in which 29% were invested. The electrical installation and the elevators together accounted for an investment of nearly 17%; and the water piping – for only about 10%. In all the other systems together, 12% of the maintenance budget was spent.

In a later paper Uhlik & Hinze (1998) report on an investigation of some 150 hospital buildings in the U.S.A. From the data it is evident that, on average, some $ 5,300 are invested annually per hospital bed in publicly owned institutions, as against $ 7,550 per bed in privately owned hospitals.

## 3 THE METHOD OF THE RESEARCH

The method of the present research consisted in a scheme of five stages: A critical literature survey, a field survey using a structured questionnaire, a statistical analysis of the data provided by the field survey, elaboration of criteria for the maintenance of the systems on the basis of the results of the statistical analysis, and the creation of quantitative indices for the efficient maintenance of multi-system buildings. The field survey included a structured interview on the lines of the questionnaire, which in its turn, consisted of two principal parts:

a. General data and a description of the character of the building. This part dealt with the data concerning the type of the facility and the mode of its operation. It included information on the area of the facility, the typical occupancy, the overall number of buildings and their characterization (floor area, purpose, and age), a comprehensive survey of the permanent maintenance department at the facility – the size of the maintenance crew, office bearers, competences, departmental budgets and their subdivision according to resources (cost of the permanent employees, the cost of materials purchased, and those spent on outside contractors, supervision of the work, and the manner of engagement of outside contractors).

b. Principal systems in the building. This part dealt with the data of the different systems in the building. These data were collected under three parameters: Present state of repair, characteristic failures, and the manner in which maintenance is carried out (checks and treatments). The systems checked in the field survey: The skeleton of the building, the envelope, interior finish, electrical installation, water, air conditioning, fire detection and extinguishing, elevators, communications and very low voltage, and finally the system of medical gases.

The data of the field survey and the statistical analysis led to the development of four indices that are based on the physical and functional states of the buildings, the maintenance inputs (manpower, budget, materials), and the organization. These indices and the manner of their application will be extensively detailed in the following.

## 4 THE FINDINGS OF THE FIELD SURVEY

The field survey comprised 17 of the principal hospitals in Israel. The main variables examined in the field survey are summed up in Table 1. It appears from Table 1 that, on average, there are in every hospital some 80,000 sqm. of built-up area and about 658 patients' beds. It follows from this that the average density in these facilities is estimated at about 8.25 beds per 1,000 sqm. built-up area. In the facilities surveyed there are in all over 700 buildings, which differ considerably from each other as regards their purpose (hospitalization, emergency ward, offices, energy, workshops, and the like), their area, and their age. The vast majority of the buildings are 30 and more years old and only a negligible minority – less than ten years. This "not very young" age of most of the buildings demands a suitable preparedness of the maintenance departments concerning their capability to deal with the great variety of treatments required for preserving these buildings in a state making them fit for use. In addition, the findings of the field survey elicited that about half the annual maintenance expenditure is devoted to the payment of the wages of the employees of the internal maintenance manpower (51%), while over a third of the annual budget is spent on the jobs of work done by outside contractors (37%), the balance – on the acquisition of materials and spare parts (12%). It was also found that the average annual cost of maintenance is about $ 38 per sq. m. or some $ 4,600 per patients' bed. That value constitutes 2.27% of the asset value of a hospital. Moreover, the span of control (the number of immediate subordinates) of the chief engineer and of the director of maintenance were examined, and it appeared that the team of the former consisted on average of 3.35 employees, the director of maintenance among them, while the latter directly controls on average about 7 employees, namely the heads of the different working crews.

**Table 1: Summary of the characteristics of the population of the field survey – Maintenance of Hospitals**

| The variable | Mean |
|---|---|
| Built-up area [$m^2$] | 79,728 |
| Number of actual patients' beds | 658.1 |
| Average density (built-up sqm. per patient's bed) | 121.1 |
| (Number of beds per 1000$m^2$ built up) | 8.25 |

| | |
|---|---|
| Annual maintenance budget ($) | 3,035,735 |
| Of which: Annual materials budget ($)[1] | 366,647 = 12.1% |
| Annual personnel budget ($)[2] | 1,533,088 = 50.5% |
| Annual outside contractors' budget ($)[3] | 1,136,000 = 37.4% |
| Mean Annual maintenance budget per built-up sqm. ($) | 38.1 |
| Asset value per built-up sqm. ($)[4] | 1,678 |
| Average annual maintenance budget (% of total value) | 2.27 |
| Annual maintenance budget per patient's bed ($) | 4,610 |
| Mean Span of Control (number of direct subordinates) | 3.35 |
| Span of Control – head-of-maintenance grade (number of direct subordinates) | 7.05 |

[1]The annual materials budget includes expenditure on the different spare parts but does not include energy costs (water, electricity, fuel, gas, etc.) and the costs of building restoration.

[2] The personnel budget was based on an assessment of the cost of work per employee - $2,500 per month.

[3] The outside contractors' budget does not include the acquisition of materials and spare parts.

[4] The asset value includes management and supervision costs, official fees, and VAT.

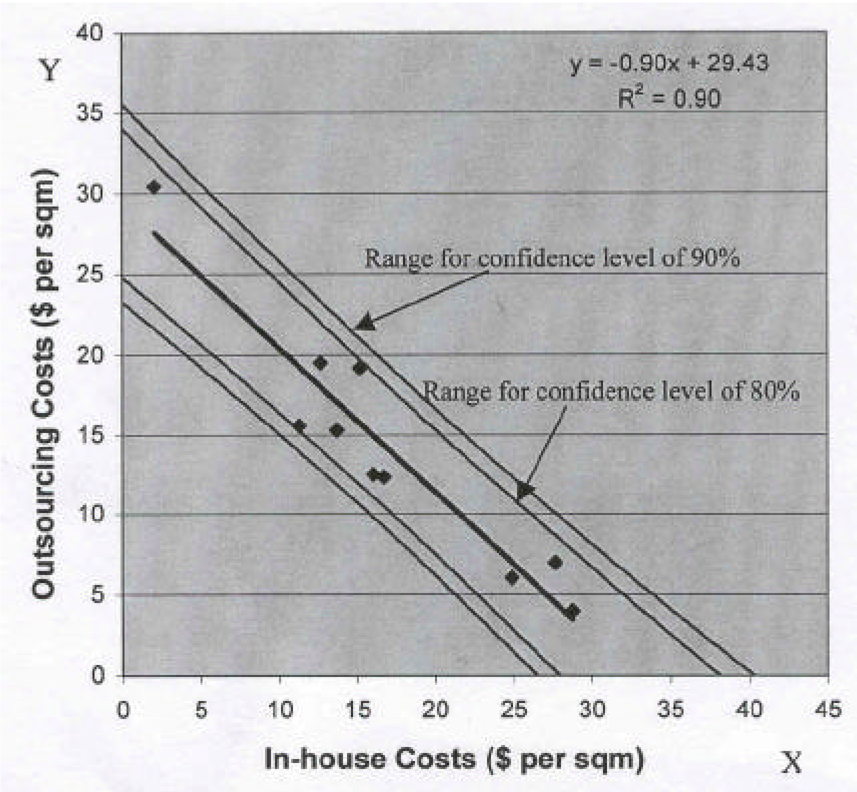


**Figure 1: Manpower Sources Diagram: Outside contractors as against in-house employees in an annual maintenance budget of 2% to 2.5% of the built-asset value.**

In Figure 1 the cost of maintenance (per sqm.) was examined as a function of the source of the work (internal employees as against outside contractors' workers-outsourcing). For this purpose the sample population was divided into two categories: Facilities with an annual maintenance budget of 2% to 2.5% of the built-asset value, and facilities with an annual maintenance budget of over 2.5% of the built-asset value. The diagram shows a linear correlation between the external and the internal sources of the work as far as the first group is concerned. The vertical axis (Y) represents the execution of work by outside contractors, the horizontal axis (X) – that by in-house employees. The main significance of the findings represented by Figure 1 and relating to the sources of the works is that any reduction by one dollar of the cost of internal employees leads to an increase of $ 0.90 per sqm. in the costs of outside contractors (outsourcing). It follows that, in these cases, investment by the maintenance department in outside manpower is preferable to investment in in-house employees. It can accordingly be concluded that the diagram reflects a considerable financial saving to be made by the employment of outside contractors for this category (the work component constitutes between 75% and 80% of the cost of maintenance).

On the other hand, the group of facilities in which the annual maintenance budget exceeds 2.5% of the built-asset value showed linear correlation between the different sources of work; but the conclusions to be drawn are that the reduction of one dollar per sqm. from the annual cost of in-house workers leads to an increase of one dollar per sqm. in the costs of outside contractors. It follows that in these cases it should not be deduced that the findings represent a financial saving through the employment of outside contractors.

When examining the annual expenditure on materials and spare parts in each of the two sub-groups it was found that in the group of facilities in which the annual maintenance costs were 2% to 2.5% of the asset value, the average expenditure on materials was about $ 4.9 per sqm. per annum. On the other hand, it was found that, in the group of facilities in which the annual maintenance costs exceeded 2.5% of the overhead costs, the average expenditure on materials was approx. $ 3.9 per sqm. per annum. From an examination of a statistical assumption concerning the two data it appeared that the two sample populations differ in the level of significance of over 95%. It follows that the findings can show a possibility to make savings in the cost of work which is derived from a higher investment in materials (which may reflect, in some of the cases, a preference for preventive over breakdown maintenance).

## 5 DEVELOPMENT OF PERFORMANCE AND BUDGETARY INDICES

The principal aim at the basis of this research was the development of quantitative management tools for examining the performance and budgeting of maintaining multi-system buildings, with the purpose of rationalizing maintenance work in accordance with the parameters discussed above. Four indices were developed with this in mind:

1. The Building Performance Index (BPI),
2. The Manpower Sources Diagram (MSD),
3. The Maintenance Input Index (MII),
4. The Managerial Span of Control (MSC).

### 5.1 The Building Performance Index (BPI)

This index expresses the efficacy of maintenance as seen in the state of the building and of the different systems in it. The rationale behind the development of this index is the need for awarding a weighted mark, from 0 to 100, to each of the building's systems, the mark $P_n$ expressing its physical and functional (performance) states. In all, 10 principal systems were defined in a building, as detailed in Table 2. The weighting of a building's system is accomplished by weighting the shares of the system's components in the costs of erection, maintenance, and replacement taken together (the Life Cycle Costs) as shown in Eq. (1). The markings are explained in the Appendix - notations and definitions, at the end of this paper.

(1)
$$W_n = \frac{\sum_{j=1}^{m}\left(C_{nj}+R_{nj}+M_{nj}\right)}{\sum_{n=1}^{10}\left[\sum_{j=1}^{m}\left(C_{nj}+R_{nj}+M_{nj}\right)\right]}$$

$$\forall n = 1,2,3,\ldots,10$$

(2)
$$BPI = \sum_{n=1}^{10} W_n * P_n$$

The weightings of the building's systems calculated in accordance with Eq. (1) are shown in Table 2.

The value of an index reflects the performance level of the building concerned: When BPI>80, the state of the building is Good and above; 70<BPI<80 indicates the state of the building as being such that some of the systems are in a marginal condition, 60<BPI<70 reflects deterioration of the building, and with BPI<60 the building is in a run-down condition. The evaluation methodology is presented in details in a different paper (Shohet, 2001).

**Table 2: The weightings of the systems of a hospital building calculated in accordance with their Life Cycle Costs.**

| The System | Weighting ($W_n$) |
|---|---|
| Interior Finish | 34.7 |
| Air Conditioning | 13.6 |
| Electricity | 12.6 |

| | |
|---|---|
| Skeleton | 12.4 |
| Water and Sanitary fixtures | 7.6 |
| Envelope | 5.3 |
| Communications & very low voltage | 4.6 |
| Elevators (Lifts) | 4.1 |
| Medical Gases | 2.6 |
| Fire Detection & Extinguishing | 2.5 |
| Total | 100.0 |

### 5.2 The Manpower Sources Diagram (MSD)

This diagram expresses the sources of the work done in carrying out maintenance – whether by an in-house work force or outside contractors (outsourcing). The rationale behind the development of this diagram is the calculation of the relationship between the costs of the various work resources (inside as against outsourcing), and the possibilities to rationalize maintenance by improving that relationship by adding to or subtracting from the various work sources.

### 5.3 The Maintenance Input Index (MII)

This index shows the maintenance inputs by means of calculating the annual maintenance budget with respect to the physical and performance state of the building. The MII provides a quantitative indication of the efficiency with which the available resources are spent. Another factor that must be taken account of when calculating the relationship between the budget and the state of the building is the age of the facility, since assets of different ages have different maintenance requirements. This factor is included in the index via an age coefficient ($Ac_d$). The maintenance expenditures include routine maintenance per component per building system ($M_{nj}$) and replacements of components at the end of the life cycle of each component ($R_{nj}$) (Eq. 3). The expenditure on replacement is dependent upon a Boolean variable B(k, d, lc, LC). (Eq. 3). B(k,d,lc,LC) receives 1 for component life cycle that falls within the decadal range, provided that the remaining life service of the building and the replaced component (LC-k*lc) greater than lc/2. The age coefficient expresses the ratio between the annual equivalent maintenance expenditure of each decade ($AME_d$) and the average annual maintenance expenditure along the entire building's life cycle ($AME_{ave}$) (Equations 4 and. 5).

(3)
$$AME_d = \sum_{n=1}^{10}\left[\sum_{j=1}^{m} M_{nj} + R_{nj} * B(k,d,lc,LC) * sp(i,N) * pr(i,10)\right]$$
$$\forall d = 1,2,...,5$$
$$N = \begin{cases} k*lc-(d-1)*10 \left| \begin{array}{l} 0 < k*lc-(d-1)*10 \le 10 \\ 50 - k*lc \ge \dfrac{lc}{2} \end{array} \right. \\ 0 \,|\, ELSE \end{cases}$$
$$B(k,d,lc,LC) = \begin{cases} 1 \left| \begin{array}{l} 0 < k*lc-(d-1)*10 \le 10 \\ 50 - k*lc \ge \dfrac{lc}{2} \end{array} \right. \\ 0 \,|\, ELSE \end{cases}$$
$$\forall k = 1,2,3,...$$

(4)
$$AME_{ave} = \frac{\sum_{d=1}^{5} AME_d}{5}$$

(5)
$$AC_d = \frac{AME_d}{AME_{ave.}}$$
$$\forall d = 1,2,...,5$$

To give an example: Assuming that the life cycle of a given building is 75 years and the annual interest payable is i=4%, it is found that the Age Coefficient ($AC_d$) for the different ages of that building are as follows: 0.58 for the first decade, 1.03 for the second decade, 1.31 for the third decade, 1.22 for the fourth decade, and 1.10 for the fifth decade. The first and second decades are mainly characterized by day-to-day maintenance, and their coefficients are therefore low; whereas the third, fourth, and fifth decades see the replacement of many components, and the values of the coefficients are therefore high. Eq. (6) describes the way in which the Maintenance Inputs Index is calculated:

(6) $$MII = \frac{AME}{AC_d} * \frac{1}{BPI} * \frac{1}{Oc}$$

The rationale behind the development of this index is the calculation of the relationship between the annual expenditure on maintenance (AME) and the resulting state of the building as expressed by the Building Performance Index (BPI), taken to be the indicator of the efficiency with which the available means are applied. The values obtained by their calculation may vary between 0 and values above it. Assuming a building at an age of 30 years, as the vast majority of buildings in the field survey, an AME of 3.4% of the Asset value, BPI of 90% expressing high performance, and a standard occupancy – i.e. Oc=1. Expression [6] yields the value of 0.48 for the aforementioned parameters.

The significances as regards the various domains being as follows:

- The range of values below 0. 5 represent a state in which the budgetary investment is low. The relevant facilities are mainly characterized by adhering to the breakdown maintenance policy, while the shares of preventive and planned maintenance as well as periodic inspections and treatments of the various systems of the building amount only to the inevitable minimum.
- The range of values between 0.5 and 0.6 describes the desired situation of a maintenance department. It reflects the efficient use of the annual maintenance budget for the various systems of the building at the level of service achieved. These facilities are characterized by executing breakdown, preventive, and planned maintenance.
- The range of values above 0.6 indicates high inputs as related to the actual results of maintenance. A high value of the index may give expression to either of two extreme situations or to a combination of the two: (1) High expenditure of maintenance; (2) Low physical performance. This range of values reflects a budgetary situation that requires managerial action with a view to rationalizing the department or the work patterns followed by it (change in the composition of the work force, preventive/breakdown maintenance, improvement of control, etc.).

At the same time it should be stressed that the ranges of the maintenance input index are liable to change, depending on the type of building: The more complicated the building, the wider will be the ranges of values, and vice-versa. Accordingly, the ranges cited above are suitable for hospital buildings, whereas the ranges for other types of building must be individually calculated -- in a similar manner, but with special reference to the particular type of building.

A further variable of which the influence is examined is the buildings occupancy (Oc) or the intensity with which the building is used. Oc [Eq. 7] depends uppon the actual occupancy of the building D as related to the standard occupancy $D_0$ determined to be 10 patiant beds per 1000 sqm, and k expresses two different ranges effects on the maintenance expenditure: below standard, and above standard occupancy. This expression was deduced following a 3 years follow-up of identical hospitalizing facilities at different occupancies. The influence of this variable fluctuates between –5% (at low occupancy) and +20% (at high occupancy) of the extent of maintenance work.

[7] $$Oc = 1 + k(D - D_0) \qquad k = \begin{cases} 0.026 \;\middle|\; 0.8 \le \dfrac{D}{D_0} \le 1 \\ 0.067 \;\middle|\; 1 \le \dfrac{D}{D_0} \le 1.33 \end{cases}$$

### 5.4 The Managerial Span of Control (MSC)

This variable is defined as the relationship between the managers and the number of personnel directly subordinate to them. The span of control is a managerial parameter, and it considerably affects the ability of a manager professionally to control his subordinates. At the same time, when determining the span of control further considerations may be introduced: The extent of the unit's activity, the complexity of the activities it takes upon itself, its geographical position, the capabilities and the qualifications of its personnel, the quality of communications within the organization, the level of uncertainty in its activities, etc. The advantages of a wide span of control find their main expression in the savings of overhead expenses, but in certain conditions a wide span causes difficulties in control. On the other hand, the advantages of a narrow span of control lie in the low quantities of day-to-day coordination and in the fact that the manager has ample time to deal with exceptional cases, but the overhead expenses are high. The rationale behind an examination of the managerial span of control is in the creation of an indicator that permits analysis of the state of the maintenance department not only from the financial (inputs) or the operational (outputs) points of view, but also as regards its organizational structure. By examining the organizational structure it will, in certain cases, identify states of lack of control and defects in communications.

## 6 APPLICATION OF THE MAINTENANCE INDICES IN HOSPITAL BUILDINGS

The model presented was implemented on the facilities surveyed in the comprehensive field survey including 17 hospital facilities. The following describes the results of the implementation of these indices.

### 6.1 The physical and performance state of a building (BPI)

The field survey pertaining the building performance encompassed 17 facilities. Table 3 depicts the results of the hospitals performance field survey. Each row represents a distinct facility, the second column shows the total value of BPI derived from expression 2, using the performance value detailed in columns 3÷12 and the weightings in Table 2. This indicates that the average grading of the systems in the hospital buildings that took part in the field survey stands at 68.9 points (on a scale of 0 to 100 with a standard deviation of 5.4 points). Based on the results of the survey it can be stated that there are systems earning relatively very high marks, high above the general average. Among these are the systems of medical gases and of fire detection and extinguishing systems. These two have in common that they are vital for saving lives, both in the course of the ongoing activities of a hospital and in its daily life (medical gases), and in times of exceptional events (fire detection and fighting). In addition, attention is drawn to the electrical and elevators systems, which were awarded higher than average marks. On the other hand, four systems are pointed out whose grading fell somewhat short of the general average, namely water, exterior envelope, air conditioning, and interior finish. The system with the lowest physical and performance grading is that of communications and very low voltage. The main system components in the building are as follows:

- Skeleton – No hospital was found in which the mark of any of the components of the skeleton was below 3 (Dangerous) on the scale of marks from 1 to 5. The typical maintenance policy with regard to this system is Breakdown Maintenance.
- Interior Finish – The physical state of the components of interior coating was found to be in the highest average grades, while the state of the floor covering was in the lowest average grades out of all the interior finish components. The maintenance policy with regard to these components is Breakdown Maintenance, and the treatment is thus administered only when necessary.
- Electricity – Failures in the electrical system are rare, most of the power failures do not originate in the internal network. The typical maintenance policy for this system is influenced by the Electricity Law, which requires periodic inspections of the various components of the system. Hence the maintenance of the electrical system is for the most part by Preventive Maintenance.
- Water and sanitary fixtures – This system is signalized by the very great number of failures and breakdowns (particularly corrosion products in the water, clogging of the sewerage, and leakage of pipes), which happen very frequently, mostly because of the long service life of the system in most of the facilities. The typical maintenance policy for the components of this system is Breakdown Maintenance, with the exception of the vital parts (such as the water heating system). In these Preventive Maintenance is the rule.
- Air Conditioning – The principal failures in this system occur in the tubing, the equipment, and the end devices. The typical maintenance policy for the air conditioning system is Preventive Maintenance. Such components as filters, compressors, pumps, and cooling towers are dealt with in all the facilities in accordance with a fixed and well-organized schedule of periodic treatments.

### 6.2 The Manpower Sources Diagram (MSD)

This diagram permits the critical examination of the way in which the efficiency of the use of human resources is determined and enables decisions to be taken with respect to changes in the composition of the personnel (in-house or external) on the strength of the field survey (Diagram 1 above). Based on this, five different models were developed with a view to increasing the human resources available for maintenance work as well as five models for reducing the resources required. From an analysis of these models it appears that the preferable alternatives for increasing resources are:

- If reliable outside contractors of known skills are available, the alternatives of increasing the share of outside contractors are preferable to the other alternatives. It will lead in certain conditions (e.g. standard level of occupancy) to considerable financial savings (of up to 25%) when the resources are thus increased and will save managerial personnel.
- If in certain trades outside contractors are not available, then it may be preferred to apply the alternatives concerning moderate additions to the budget, both in the section on outside contractors and in that of in-house employees, depending on the situation with regard to outside contractors or, in more exceptional cases, the share of in-house personnel only is to be increased.
- On the other hand, the preferable alternatives to reducing the budgetary resources of the maintenance department would be:
- If reliable outside contractors of known skills are available, the alternative of considerably reducing the budget of in-house personnel seems preferable to the other alternatives. It makes potential savings both in the financial costs of maintenance and in administrative personnel.
- If outside contractors in certain trades are not available, the alternative of reducing the budget for outside contractors can be used together with a budget reduction, preservation, or even increase, for in-house employees, having regard to the situation with respect to outside contractors.

**Table 3: Building Performance Index (BPI) in a field survey of 17 hospital buildings in ISRAEL**

| Serial No. | BPI | Skeleton | Exterior Envelope | Interior Finishes | Electricity | Water & Water-Waste | HVAC | Fire Protection | Elevators | Communication | Medical Gases |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | **80.4** | 94.0 | 88.3 | 86.0 | 83.3 | 72.9 | 60.0 | 75.0 | 61.4 | 41.7 | 100.0 |
| 2 | **78.3** | 68.5 | 71.0 | 80.2 | 91.7 | 79.2 | 70.0 | 75.0 | 87.1 | 41.7 | 100.0 |
| 3 | **74.1** | 90.0 | 79.3 | 77.3 | 58.3 | 58.3 | 65.0 | 100.0 | 74.3 | 58.3 | 100.0 |
| 4 | **74.1** | 90.0 | 95.8 | 70.0 | 58.3 | 85.4 | 62.5 | 75.0 | 87.1 | 58.3 | 100.0 |
| 5 | **72.6** | 72.0 | 35.3 | 74.6 | 91.7 | 58.3 | 66.7 | 75.0 | 74.3 | * | 100.0 |
| 6 | **71.4** | 82.0 | 79.3 | 77.3 | 58.3 | 70.8 | 52.5 | 75.0 | 74.3 | 41.7 | 100.0 |
| 7 | **69.4** | 67.5 | 53.0 | 77.7 | 58.3 | 66.7 | 62.5 | 75.0 | 82.9 | 25.0 | 100.0 |
| 8 | **67.4** | 78.0 | 54.5 | 56.1 | 83.3 | 60.4 | 72.5 | 100.0 | 70.0 | 25.0 | 100.0 |
| 9 | **67.2** | 68.5 | 54.5 | 59.0 | 91.7 | 58.3 | 72.5 | 75.0 | 57.1 | 41.7 | 100.0 |
| 10 | **66.1** | 72.0 | 67.7 | 56.1 | 66.7 | 79.2 | 65.0 | 100.0 | 82.9 | 25.0 | 100.0 |
| 11 | **66.1** | 60.0 | 62.0 | 55.4 | 91.7 | 58.3 | 71.7 | 75.0 | 70.0 | * | 100.0 |
| 12 | **65.4** | 72.0 | 63.1 | 54.7 | 91.7 | 52.1 | 60.0 | 100.0 | 70.0 | 33.3 | 100.0 |
| 13 | **64.7** | 84.0 | 62.0 | 53.3 | 58.3 | 75.0 | 67.5 | 75.0 | 78.6 | 41.7 | 100.0 |
| 14 | **64.7** | 78.0 | 63.5 | 56.1 | 58.3 | 70.8 | 70.0 | 75.0 | 70.0 | 41.7 | 100.0 |
| 15 | **63.7** | 66.0 | 59.3 | 50.5 | 91.7 | 58.3 | 57.5 | 100.0 | 74.3 | * | 100.0 |
| 16 | **63.3** | 72.0 | 82.7 | 56.1 | 66.7 | 60.4 | 58.1 | 100.0 | 48.6 | 41.7 | 100.0 |
| 17 | **62.0** | 66.0 | 33.8 | 47.7 | 91.7 | 58.3 | 67.5 | 75.0 | 82.9 | 33.3 | 100.0 |
| Mean | **68.9** | 75.3 | 65.0 | 64.0 | 76.0 | 66.1 | 64.8 | 83.8 | 73.3 | 39.3 | 100.0 |

* Not available.

### 6.3 Maintenance Input Index (MII)

This index can provide the decision makers (chief engineer or maintenance director) with a quantitative tool for determining if the maintenance inputs indicate their efficient use relative to the physical and performance state of the building. Table 4 describes a comparative case study of two extreme states of the MII.

Table 4 presents two hospitals, A and B. In Hospital A the annual cost of maintenance is $34.8 per sqm. (about 2.07% of asset value); the average age of the buildings is 35 years, the Age Coefficient ($AC_d$) with reference to the age of the buildings (which expresses the relative extent of maintenance work in accordance with the decade in which the buildings are) is 1.22 (as explained above); and the performance state of the buildings is BPI = 78.2. That value reflects a relatively good state of repair of the buildings' systems (despite the high grading, the communications and very low voltage system is an exception due to its very low state of repair). The density coefficient reflects the effect of the occupancy level as compared with a standard level of occupancy. The Density coefficient value for this hospital is 0.95 representing a relatively low level of occupancy (7.6 patients' beds per 1000 sqm. built-up compared with a standard of 10). The maintenance input index derived from these data is 0.39 – lower than the average index of 0.5. The index expresses the efficient use of resources and of personnel, as well as a certain shortage of resources. On the other hand, in Hospital B the annual cost of maintenance is $44.6 per sqm. (about 2.66% of the asset value); the average age of the buildings is 44 years, in other words the Age Coefficient with reference to the age of the buildings is 1.10, and the state of the buildings' performance capacity is BPI = 64.7, indicative of the buildings' deteriorating state. The density coefficient for this hospital is 0.99 representing a nearly standard occupancy (9.5 compared with a standard of 10 patients' beds per 1000 sqm. built-up). As a result, the Maintenance Input Index (MII) is awarded a value of 0.64, a high value indicating the high level of expenditures relative to the low performance state of the buildings. The conclusions that can be drawn from this analysis are that despite the better physical and functional condition of the buildings of facility A compared with that of facility B, it is entitled to a small increase in resources. In facility B, on the other hand, the physical and functional condition of which is low, the use of in-house and external manpower resources must be significantly rationalized – a change that will bring about a reduction in costs and an improvement in results.

**Table 4: Application of the Maintenance Input Index in two hospitals**

| The Variable | Hospital A | Hospital B |
|---|---|---|
| Annual cost of maintenance ($\$/m^2$) | 34.8 | 44.6 |
| Average age of the buildings (years) | 35 | 44 |
| Repair coefficient for the age of the buildings | 1.22 | 1.10 |
| Building performance index (BPI) | 78.2 | 64.7 |
| Maintenance input index (MII) | 0.39 | 0.64 |

### 6.4 The managerial span of control (MSC)

As regards the chief engineer's span of control it was found that on average 3.35 employees are directly subordinate to him. However, the standard deviation of this value is very high, which points to the great divergences between the maintenance departments surveyed. When the span of managerial control of the director of maintenance was examined, it turned out that on average 7.05 persons are directly given to his control, hence the divergence between facilities is small relative to the preceding value.

The data elicited by the field survey indicate that most of the maintenance departments in the hospitals operate as mechanical units, the chief characteristics of which are: A rigid hierarchical structure with a restricted capability to compose different work crews for different types of projects; and most of the activities are carried on under formal rules and procedures. An organization of this kind can operate if the following environmental conditions exist: A mid-sized to large organization, a stable work environment with prolonged and established activity, and standard work processes. On the other hand, other maintenance departments were observed that operate as organic organizations, mainly characterized by: A flexible hierarchical structure, with the possibility of making organizational changes in accordance with the particular tasks in hand, a special task devolving on the assistant staff (in this case the engineers of the maintenance department, mainly thanks to the flexible structure). Such an organization can operate if the following environmental conditions are fulfilled: A dynamic environment that is subjected to many and frequent changes, the tasks to be performed include complex actions (or complicated projects), and use is made of advanced technologies. Managing maintenance of multi-system buildings demands interdisciplinary activity that sometimes combines a number of trades or professions simultaneously, for instance air conditioning, energy, communications, etc.

## 7 DISCUSSION AND CONCLUSIONS

The main points in the rationalization of maintenance management, as found in the field survey and forming part of the Integrated Maintenance Model developed in the present research, are as follows, according to the various categories examined:

### 7.1 Maintenance Policy

1. The efficiency of maintenance can be increased in most hospitals by consistently carrying out periodic inspections. Preventive and planned maintenance can be introduced with simple means and at low financial expense.
2. The combination of in-house employees and workers of external contractors in the different systems of a building can lead to many advantages derived both from the work of in-house personnel (availability, reliability, and loyalty) and from the services of outside contractors (cost). Obviously in both sources of work the high quality of the workers must be insisted on.
3. It was found that the service life of hospital buildings in Israel is 70-80 years. The research did, however, also discover that in these buildings the annual cost equal to maintenance and the replacement of the components of a building is higher by about 20% than the corresponding cost of buildings with a life expectancy of 50 years; in other words continuing their preservation beyond a range of 60 years is not economical.

### 7.2 Maintenance Budgeting

4. The Maintenance Input Index permits assessing the efficiency of the use of the resources set aside for maintenance, having regard to the existing level of service, the required level of service (output), the weighted age of the buildings on the campus, and the typical occupancy of campuses.
5. Concerning the connection between the level of investment in spare parts and materials on the one hand and the annual cost of maintenance per sqm., on the other hand, it was found that an inverse relationship exists between the two variables, which is to say that increasing the input of materials and spare parts is attended by a reduction in the overall cost of maintenance.

### 7.3 Organizational Structure

6. It transpired from the research that another five to six engineers in the fields of electricity, air conditioning, water and plumbing, communications and computers, and civil engineering, as well as a director of maintenance, serve under the chief engineer. Subordinate to the director of maintenance and his deputy are seven to ten work crews in the various fields. The size of the organizational structure can be changed to make it accord with the size of the hospital and as required by the existing policy relative to the sources of the work (in-house or external).
7. It was found that in most of the hospitals the structure of the work teams is very rigid. It seems that a more organically flexible structure will make possible the combined action of several such teams and the cooperative work on specific projects. This is a necessary and desirable pattern of work in multi-systems' buildings.

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## 9 APPENDIX – NOTATIONS AND DEFINITIONS

i - The annual rate of interest

n - The system of a building (in all 10 systems were defined)

j - A component in a system

m - The total number of components in a system, n

$W_n$ - The weight of the system in the Building Performance Index (BPI)

$C_{nj}$ - The cost of installing a component, j, in a system, n

$R_{nj}$- The cost of replacing a component, j, in a system, n

$M_{nj}$ - The annual cost of the maintenance of a component, j, in system, n

MSD- Manpower Sources Diagram

MII - Maintenance Input Index

AME - The annual expenditure on maintenance ($\$/m^2$)

Oc – Occupancy coefficient

D – Actual occupancy

$D_0$ – Standard planned occupancy

BPI - Building Performance Index

$P_n$ - The performance index of a system, n

d – Decadal index along the life cycle of the building

$AME_d$ – Annual Maintenance Expenditure of decade d

$R_{nj}$- Expenditure for Replacement of component j in system n

N – The time (in years) in decade d in which the component is replaced

Sp(i, N) – The Present Value coefficient for a single payment at time N

Pr (i, 10) – The equivalent annual value coefficient transforming a present payment into a series of annual uniform payments along a period of 10 years.

lc – Standard Life Cycle of component

LC – Building's life Cycle

K – Component's life cycle index along the building life cycle (LC)

B (k, lc, LC) – A Boolean variable for decision on the replacement of component with life cycle – lc, in a building with a life cycle – LC, at the end of the $k^{th}$ life cycle

$AME_{ave}$ – Average decadal Maintenance Expenditure

$AC_d$ – Age coefficient for decade d

# The Belcam Project: A Summary Of Three Years Of Research In Service Life Prediction And Information Technology

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**Summary: The objectives of the Building Envelope Life Cycle Asset Management (BELCAM) Project were to develop techniques to predict the remaining service life of building envelope components and procedures to optimize their maintenance. Six enabling technologies were identified as critical to the tasks: service life prediction, life cycle economics, risk analysis, maintenance optimization, and information technologies. Roofing systems were chosen as the domain for the "proof of concept" of the techniques and procedures. Information technology was to be used extensively in the course of the project. During the three-year term of the project, data were collected on 2800 roof sections from a wide range of systems and climatic regions across Canada. Data in this paper are presented based on age, material type, geographic location and condition of the roofing sections. Markov Chain modeling was used to predict the change in conditions of representative samples; deterioration curves were generated to predict the change in condition, and remaining service life of specific components of the roofing system could be estimated from these data. The first objective was accomplished through these activities. The project then developed techniques to estimate the life cycle costs for different maintenance strategies and to estimate the risk of envelope failure. Multi-objective optimization was used to prioritize planned maintenance, based on maximizing condition, while minimizing risk of failure and cost of repairs; thereby attaining the second objective of the project. A prototype, graphical, decision-support tool, developed as a result of this research, is described. A main goal of the project was to utilize information technology to a heavy degree in data collection, analysis and display. However, slow developments in the field of standards for product models in the building envelope and asset management domains (i.e. Standard for the Exchange of Product Model Data - STEP and International Alliance for Interoperability - IAI) prevented the development of frameworks for storing and sharing these data. There is a need for continued research in these areas. This research will continue for an additional three years in collaboration with four Canadian universities; in course of this research additional roofing data will be collected and industry foundation classes (IFC) will be investigated as models for data exchange.**

**Keywords: Service Life, Maintenance Management, Roofing Systems, Visualization, and Information Technology.**

## 1 INTRODUCTION

The Building Envelope Life Cycle Asset Management (BELCAM 2001) project was conceptualized over five years ago (Lacasse & Vanier 1996; Vanier & Lacasse 1996). It was initiated collaboratively by the National Research Council Canada (NRCC) and Public Works and Government Services Canada (PWGSC) in March 1997 and was based on these organizations' extensive research on durability and service life prediction. One overriding goal of the project was to investigate service life of building components in a holistic fashion, rather than just investigating material properties.

### 1.1 Project objectives

Initial investigation in the service life research for BELCAM (Lacasse & Vanier 1996) indicated that few research projects looked at components systemically, with very few of the projects even mentioning economic issues, risk factors or user requirements. That is, pre 1990's the body of research literature on durability relied principally on material properties to determine service life. BELCAM was one of the first attempts to investigate the service life of multi-component systems using visual inspection techniques. It also identified a number of other enabling technologies affecting the service life of components including: life cycle economics, risk analysis, maintenance management, user requirement models and information technologies (Vanier & Lacasse 1996).

The objectives of the BELCAM project were: (1) to develop techniques to predict the remaining service life of building envelope components and (2) to develop procedures to optimize their maintenance management. Since the domain of the entire building envelope was too large for the budget of the project, it was decided to concentrate on one specific domain in a vertical rather than a horizontal implementation scheme. Roofing systems were chosen as the domain for the "proof of concept" of the techniques and procedures based on the high cost of roofing maintenance, repair, and renewal and the availability of a roofing system condition assessment survey (CAS) technique (Bailey *et al* 1989). It was also planned to use information technology extensively in the course of the project based on NRCC's experience in this field and the pre-eminence of this technology in business at that time.

A consortium was formed by NRCC and PWGSC to finance the proposed research endeavour. The first phase of the BELCAM project lasted three years ending in March 2000. The total project costs, including "in-kind" contributions were approximately $ 2.0 million CDN ($ 1.0 CDN = $0.67 US = $0.80 Euro). The BELCAM Consortium partners are listed in the Acknowledgements at the end of this paper. A small team of four to five researchers participated full-time in the project and a handful of inspectors collected roofing data at various locations across Canada.

### 1.2 Six enabling technologies

As identified earlier, initial investigation into the field of service life prediction identified six enabling technologies that were critical to the attainment of the aforementioned objectives of the BELCAM project: maintenance management (Lacasse & Vanier 1996), service life prediction, life cycle economics, user requirement models (Vanier *et al* 1996), risk analysis (Lounis *et al*, 1998) and information technologies (Vanier 1998). Each of these enabling technologies was required by practitioners to estimate the remaining service life of components and to assist in maximizing the return on maintenance expenditures. Each enabling technology, in its own way, contributes a different facet to the service life maintenance management of building envelope components. For example, building owners are naturally interested in the material properties of their components; however many of their components and systems were being replaced prematurely owing to poor maintenance, changes in user requirements, high life cycle costs, or increased risk of failure. In addition, there were few metrics to determine the contributions of each one of these facets of life cycle asset management.

## 2 BELCAM METHODOLOGY

Efforts were initiated to investigate each of these six enabling technologies. In the course of the project, a number of papers have summarized these efforts to date (BELCAM 2001). Without going into the exhaustive detail outlined in these related papers, the project attempted to reach the aforementioned goals with the following steps:

1. Develop a life cycle asset management framework,
2. Select condition assessment survey (CAS) protocol,
3. Collect data meeting BELCAM criteria,
4. Centralize collected data,
5. Develop a Markov Chain-based service life prediction model,
6. Generate simplified deterioration curves,
7. Develop risk model, and
8. Develop decision support software to display graphically the existing and forecast data.

### 2.1 Condition assessment survey protocol

The following subsections describe the material and methods used to collect data for BELCAM.

#### 2.1.1 Data collection tool

Following an evaluation of available roofing inspection and maintenance management software (Vanier *et al.* 1998), *MicroROOFER vers.1.3* (Bailey *et al*. 1989; MicroROOFER 2001) was selected as the base data acquisition software for the project. The software evaluation examined four commercially available packages relative to: ease of use, required minimum hardware configurations, operating platforms and database structure, technical and reporting features, as well as their ability to interface and link with other applications. The evaluation also did not consider nine roofing software tools from the review and from usage in the project because they did not meet BELCAM essential criteria for condition assessment surveys. This short list also included a number of programs that were proprietary to the developing company and not commercially available.

MicroROOFER was considered to be the most comprehensive software package of the products studied in the domain of condition assessment and was selected to host the data for the regional surveys.

### 2.1.2 Electronic data capture on site

A review of existing data collection technologies, both hardware and software lead to the expansion of the standard data collection framework to include electronic data collection on site. The Fujitsu 1200 Stylistic™ pen-based computers running Microsoft Windows 95™ operating system was selected owing to its cost, availability, and robustness. Other overriding factors for the selection of this equipment included the important issue of visibility of the screen under extreme lighting conditions (transflexive screen with gray scales and backlighting). The units are valued at approximately CDN$ 7000.

### 2.1.3 Digital images

The utility of digital images was noted and, although not explicitly a component of the standardized data collection package, each agency was encouraged to use these digital visual-recording techniques as part of their *"survey kit"*. The digital cameras were used primarily in the inspection process to record the actual condition of the roofing distresses.

### 2.1.4 BELCAM protocols

An examination of the existing data fields of MicroROOFER (2001) and a comparison to the BELCAM information requirements revealed numerous data gaps. In order to assure that adequate information was collected on all aspects of roofing performance (design and as-built conditions, material and workmanship quality, and condition of structural elements), these additional BELCAM data. In all cases, the identified BELCAM protocol requirements were recorded in existing remarks fields in MicroROOFER data requirements were identified and appropriate storage locations within MicroROOFER were selected as a repository for the BELCAM.

The BELCAM "Roofing Condition Assessment Survey" (RCAS) methodology assisted in obtaining consistent and easily interpreted information by detailing recommended methods of data collection and recording. A web-based *On-line RCAS Manual* provided a description of the MicroROOFER (2001) program requirements, as well as the BELCAM data requirements (Lounis *et al* 1999). The RCAS files were also made available in electronic form for easy access on the pen-based systems. Many of the recommended RCAS procedures offered innovative methods for inspection, and recording information (e.g. determination of deck type and condition, quantification of extent of ponding, etc.). Use of the on-line RCAS manuals, coupled with the pen-based systems and digital cameras, reduced learning time required to conduct inspections and minimized the time between inspection and data entry.

BELCAM's RCAS provided the regional data collection teams with easily accessed information on roofing defects, their definitions, how and what to inspect as well as a consistent methodology to record specific roofing distresses. It was also supplemented with the electronic versions of the related MicroROOFER manuals, both on the web and on the pen-based systems.

## 2.2 Data gathering

During the three-year term of the project, data were collected on over 2800 roof sections from a wide range of systems and climatic regions across Canada. Because of the usage MicroROOFER as the only data collection tool, there was a standard data format and versioning from the data collected in the field.

MicroROOFER runs under Microsoft Access®. The BELCAM regional survey sites forward their complete database file (*.mdb) annually to the authors by email. A macro created in MS Access®, exports the required MS Access® data to a number of text files (*.txt). These files are imported into the BELCAM national database, running on 4$^{th}$ Dimension®. These data include inventory information (location, roof type, areas, roof and building age, sketches, etc.), condition of the membrane, insulation and flashings, and the type, severity, and quantity of the distresses. Although the first phase of the project is complete, data collection is continuing for another three years under the auspices of a Natural Sciences and Engineering Research Council grant (NSERC 2001).

## 2.3 Data analysis

The MicroROOFER methodology uses the quantification of visible distresses to determine the condition rating of the roof flashing and membrane. The condition rating of the insulation is determined from core sampling and evaluation of the percentage of wet insulation (Bailey *et al* 1989). These condition ratings are mapped to condition states adopted in a discrete Markov chain model. For example, a MicroROOFER rating of 85% to 100% is mapped to BELCAM State 7 or excellent condition and a MicroROOFER rating of 0% to 25% is in a failed state or BELCAM State 1 (Lounis *et al* 1999).

### 2.3.1 Roofing condition data

Data were received from across Canada, as shown in Fig. 1. The small circles represent the central location for the regional surveys and the large circles show the area extent of the data collection. Since the data collection tool is standardized, data from the regional survey sites are highly compatible. These data were grouped into seven climatic zones (one region has three climatic zones, one with higher rainfall and one that is colder).

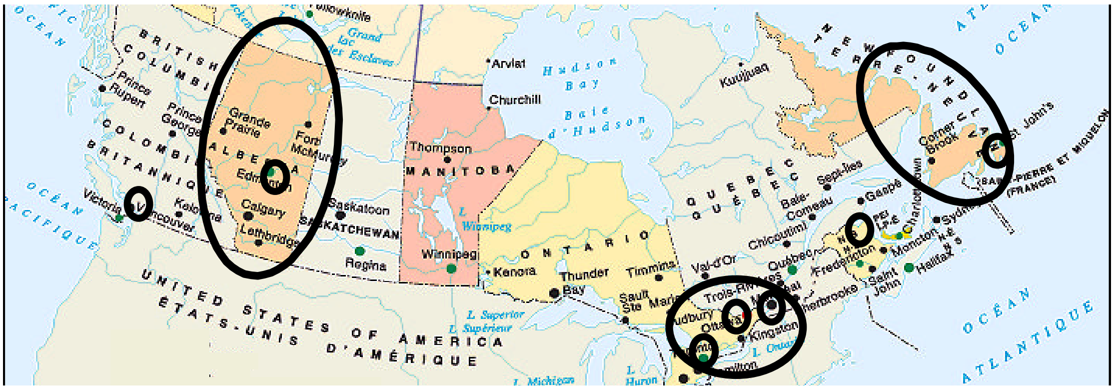


**Figure 1. Map of BELCAM regional surveys.**

These regional surveys primarily include roofs owned and operated by federal and provincial governments, crown corporations or publicly funded universities. The data are from roughly 600 buildings in approximately 15 cities or towns. Each building typically has a number of roof sections. In total, roughly 13,000 individual visual distresses were identified, classified and quantified (Kyle & Vanier 2001) on the 3000 roof sections.

Fig. 2 shows the distribution of representative roofs in-situ by material type. The vast majority of sections in the survey were BUR or multi-layer application of modified bitumen membranes. While the population profile is representative of the typical Canadian situation, it does not reflect the membrane type profiles for new roofing construction (CRCA 1995). In consideration of the industry's increasing usage of thermoplastics (PVC & TPO) and the necessity to have reliable in-situ performance data, future BELCAM survey samples will include higher percentages of these membrane materials.

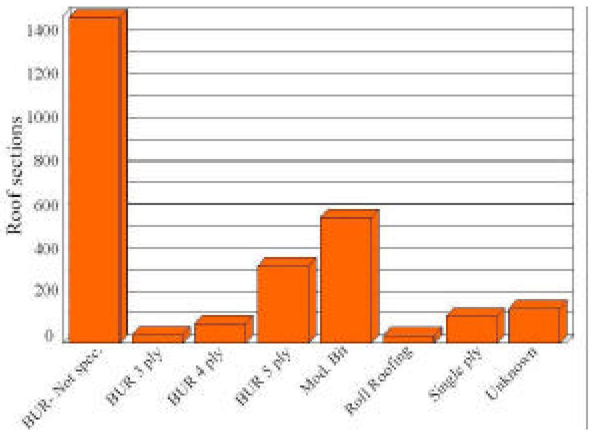


**Figure 2. Histogram of membrane type.**

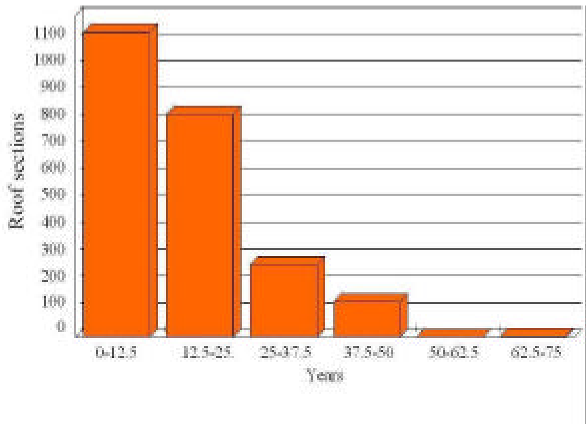


**Figure 3. Histogram of membrane age.**

As the BELCAM survey sites typically performed visual inspections on their total portfolio, the roof ages reflect the age of their building stock as well as the extent of renewal. Figure 3 shows the distribution of membrane age, grouped into age classes. It must be kept in mind that the building owners in this survey are government or para-government organizations and, as such, are knowledgeable owners and try to optimize their life cycle costs.

Figure 4 illustrates the assessed condition of the membranes in the survey sample. In general, the condition of the roof sections in this study is very good. As well, the high percentage at relatively young ages is an indication of a good renewal rate. This particular characteristic is believed to be because of the high percentage of government and para-government owners in the survey. Figure 5 illustrates the assessed condition of flashings. Histograms are also available for roof insulation (Vanier & Kyle 2001).

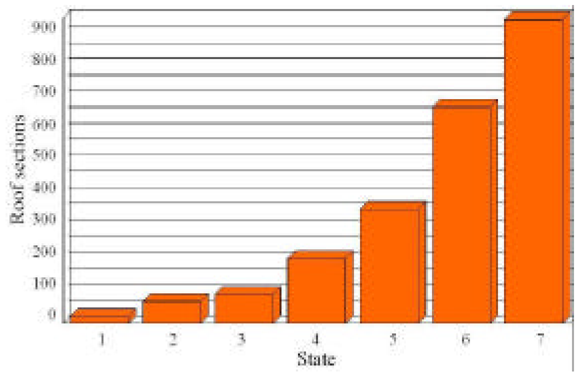


Figure 4. Histogram of membrane condition

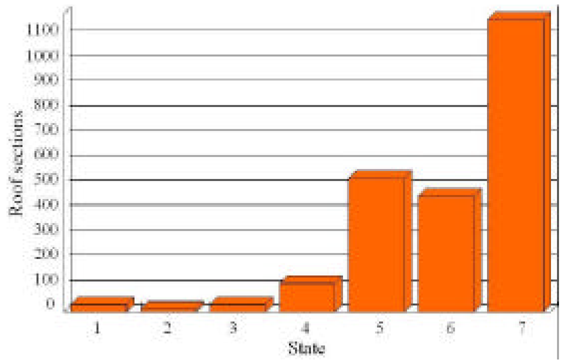


Figure 5. Histogram of flashing condition.

Figures 6 and 7 show the condition of the roof membranes and flashings respectively on a regional basis. One can readily see the direct relationship between age and condition in these two figures.

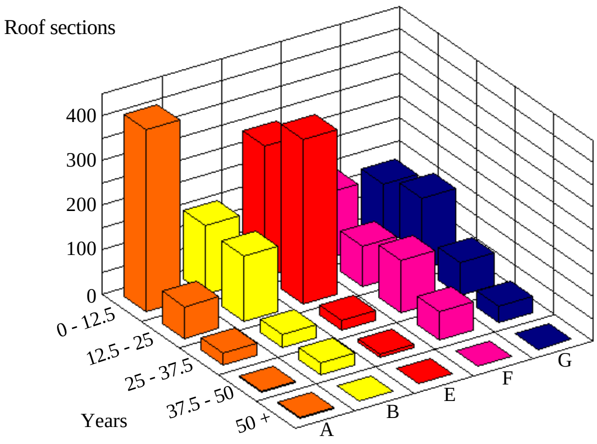


Figure 6. Histogram of age class and region.

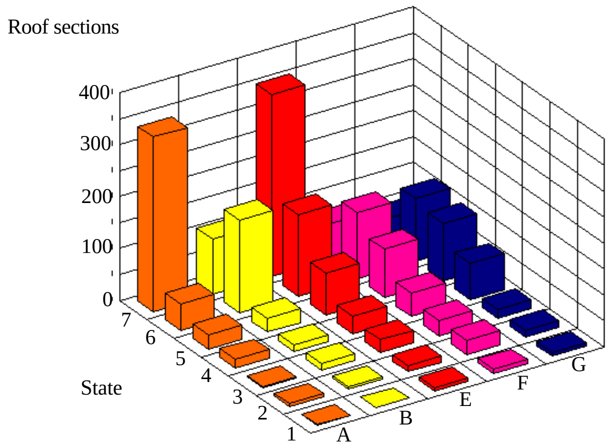


Figure 7. Histogram of condition and region.

2.3.2 Roofing distress data

Figure 8 presents the percentage of distresses for the different membrane types. The predominant systems in the project are built-up roofs (BUR) and modified bituminous (Mod. Bit). Figure 9 displays the severity breakdown for the entire sample and Fig. 10 plots the percentages of the three severity types of multiply roofing membranes for each age class. Generally, low severity distresses become high severity distresses over time; more detailed data can be found in the related paper (Kyle & Vanier 2001)

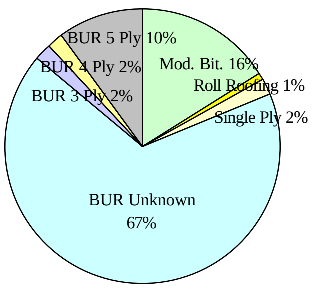


Figure 8. Distresses.

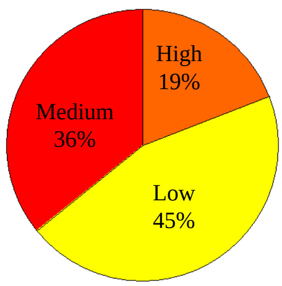


Figure 9. Distress severities.

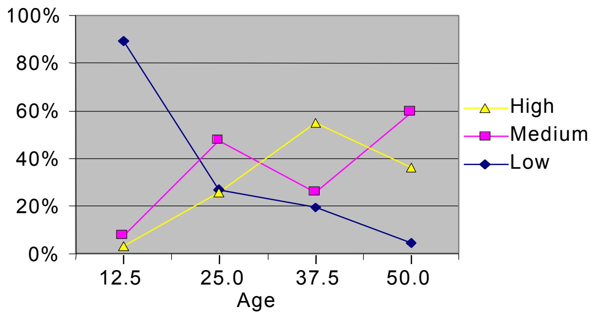


Figure 10. Distress severity and roof age.

## 2.4 Service life prediction modeling

A Markov Chain model was used to develop a roofing deterioration model that predicts the change in conditions of representative samples. Deterioration curves were then generated and plotted to predict the change in condition. The remaining service life of the roofing system could be estimated from these data (Lounis *et al* 1998). Figure 11 displays average deterioration curve of BUR roofs for all regions across Canada. Similar curves were developed for Mod. Bit. and single ply roofing, as were regional curves for generalized conditions and for specific membrane types.

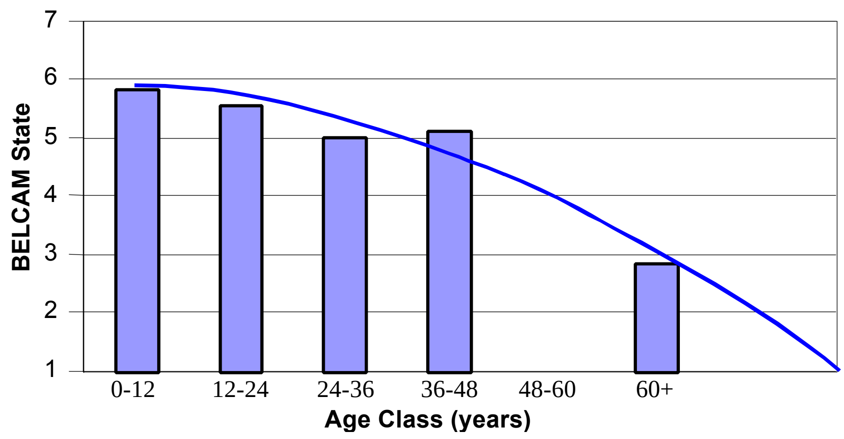


**Figure 11. Average condition curve of built-up roofing membrane**

### 2.5 Maintenance prioritization

The project team developed techniques to calculate life cycle costs of alternative maintenance strategies based on standards (ASTM E917) and estimate the risk of envelope failure (Lounis *et al.* 1999). Multi-objective optimization is used to prioritize the planned maintenance, based on a trade-off analysis of maximizing condition, while minimizing risk of failure and cost of repairs (Lounis & Vanier 2000).

### 2.6 Decision support software

A two-dimensional, graphical prototype tool christened Visualizer was developed to demonstrate the decision support capabilities; it is described elsewhere (Kyle *et al.* 2002). An original goal of the project was to utilize information technology extensively in the collection, analysis and display of data. This was possible for the data collection and analysis, but it was only possible for a limited number of applications related to the roofing domain. Figure 12 displays the use of Visualizer to import a background image as well as the data files from MicroROOFER. Figure 13 shows how the graphical elements on the imported image (white rectangle at top left of figure) can be linked to the imported data file from MicroROOFER (menu pick in the displayed dialogue box). The gray scales in Fig. 13 represent numerical data such as roof condition or cost of repair; for example, darker shades of gray could either represent inferior condition or expensive repairs, respectively.

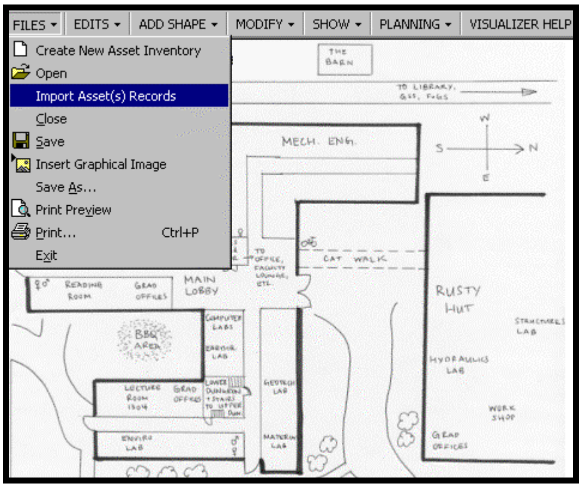


**Figure 12. Import option for images and files.**

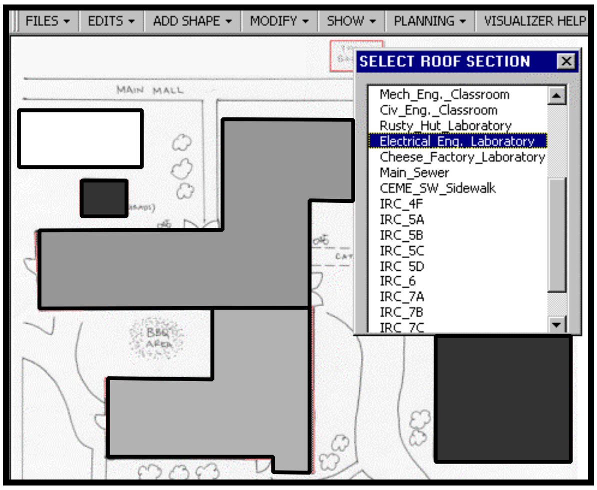


**Figure 13. Linking database records to graphics**

## 3 DISCUSSION

The discussion concentrates on the individual components of the project identified in the second section of the paper. Issues regarding the overall project and its major facets are presented in the Summary and Conclusion section.

### 3.1 Condition assessment survey protocol

The current BELCAM protocols and database provide the capability to assess a large data set and to examine various aspects on a population-wide basis including regional differences in component condition, age distribution and material selection. The condition of the roof sections in this study is generally very good. As well, the high percentage of relatively young age roofs is an indication of a good renewal rate and progressive owners. While the population profile is small, it is representative of the typical governmental existing building stock, but it does not reflect the type profiles for new roofing construction (CRCA 1995).

The data collection for the project proved difficult both to initiate at individual regional sites and to continue in subsequent years; the asset managers of the portfolio had to be convinced of the merits in order to secure funding for the inspections. The cost of the software was miniscule compared to the labour required to inspect the roofs. The cost of inspection for the BELCAM regional surveys is not available and could not be considered representative of trained and certified CAS inspectors; however, companies currently performing a similar service (including core samples) are charging CDN$ 0.13 per square metre (Sydneysmith 2001). More roof data will be collected in the upcoming two years by BELCAM team in an affiliated project (NSERC 2001).

The pen-based computers assisted in data collection, but some inexperienced users encountered problems mainly related to use of character recognition software; however, the equipment was useful by dedicated and trained users. It was noted by some organizations that the equipment would be expensive for agencies with many inspectors.

### 3.2 Data gathering

The amalgamation of the data from the seven organizations participating in the BELCAM project proved to be quite simple. Since the data collection tool has been standardized to one version of MicroROOFER, the extracted data was consistent; the same holds true for different versions of MicroROOFER. However, the majority of other computerized maintenance management software (CMMS) in the roofing domain does not save the mandatory data for the service life research. Some extraneous records were found to be present in some of the data transmissions, but these were easily culled.

### 3.3 Data analysis

The majority of the reported distresses occurred on BUR roofs, with roughly one-third being flashing related. The flashing distresses for modified bituminous installations accounted for approximately 20% of those observed. Single-ply installations, however, had more reported flashing defects than membrane issues. While the single-ply roof sample is small, the high percentage of flashing distresses may indicate a lack of familiarity with some material-specific flashing design or construction concerns not at issue with more conventional (multi-ply) practices.

The severity of defects is typically expected to worsen with time. The observed severity populations confirm this. However, it must be noted that there is a significant difference in the nature of change of the severity from one distress type to another, i.e. the shapes of the percentage population curves for low, medium, and high severity are distress specific.

### 3.4 Service life prediction modeling

Industry will require years if not decades of data collection to have enough data to develop Markov Chain-based service life models in all the important domains. This three-year project barely produced enough data to generate service life curves for the generalized cases for specific types of low slope roofs. In addition, insufficient multi-year data was collected in the term of the project for two principal reasons: (1) there is very little change in the performance in this time frame to warrant annual inspections and (2) inspections are too expensive to carry out annually for every asset type in a large portfolio. In place of multi-year data, a technique using random sampling of data was used to generate the required deterioration curves.

In addition to the data analysis using Markov Chain, a small research project using case-based reasoning (CBR) was completed. Because of the dearth of multi-year data, the CBR results cannot be used at this time. It is hoped to resume this effort in the future using multi-year and repair data in related projects (NSERC 2001).

As indicated in the previous subsection, there was not adequate multi-year data and there was not enough data in specific classes of roofing types or regions to provide adequate service life prediction for these cases. As more users begin to collect data for condition and as more performance metrics are developed, there will be more ready access to this type of data for service life prediction.

### 3.5 Maintenance prioritization

The conceptual model for maintenance priorization was developed in the course of the project (Lounis *et al* 1999 2000). However, the maintenance priorization was not fully implemented in the decision-support software owing to time concerns and insufficient resources. A simplified engineering model was developed and implemented in Visualizer to identify the maintenance projects meeting the criteria of “worst condition”, “oldest assets” or “most inexpensive”. The theoretical possibilities of this implementation is presented in a related paper at this conference (Kyle *et al* 2002).

Decisions regarding the prioritization of maintenance projects must take into account three objectives: condition, cost and risk (Lounis *et al* 2000). An “engineering” model for risk assessment was implemented in Visualizer to identify the potential cost of repair of the envelope and the cost of collateral damage to the building and occupants. The risk of failure is dependant on the type of facility, which is based on the building category code obtained from MicroROOFER. Each building category code is assigned a collateral damage (CD) code using heuristics about that type of facility. CD values range from 1 for low damage to 7 for high damage; the potential collateral damage is then equal to $10^{CD+1}$. This simplified approach is used to determine the "relative" risk of failure amongst a number of assets and to position that risk in relationship to the cost of maintenance and repairs.

### 3.6 Decision support software

Maturity in the field of interoperability and product modeling has been very slow over the past three years. This slow developments in the field of standards for product models in the building envelope and asset management domains, namely the Standard for the Exchange of Product Model Data (STEP 2001) and International Alliance for Interoperability (IAI 2001) prevented the development of suitable "product model" frameworks for storing and sharing these data.

In place of the preferred implementation of product model technology, interfaces were developed in the BELCAM project using "off-the-shelf" software applications such as Microsoft Excel® and VBA. It is hoped that the next stage in BELCAM (NSERC 2001) will include the development of industry foundation classes (IFC) to address interoperability issues.

## 4 RECOMMENDATIONS

The following recommendation are made to progressive researchers and practitioners interested in the field of life cycle asset management:

#### 4.1.1 Condition assessment surveys (CAS)

- There is a need for more CAS performance metrics in the construction area. Reliable and rigourous performance evaluation cannot take place without CAS. However, CAS's are difficult to develop and to validate. Researchers and practitioners should develop CAS performance metrics and try to standardize them.
- There is an increasing interest in CAS in the construction industry. "Early adoption" practitioners should use this technology as soon as possible.
- Although the type of data collected and the amount of detail were mandatory for the research, simplified performance evaluations should be used until validated CAS are available.

#### 4.1.2 Materials Testing

- Future research should examine the correlation between visual inspection data and laboratory testing. This endeavor will increase the available samples and lead to refined statistical linkages between deterioration of membrane and insulation.
- In consideration of the industry's increasing usage of thermoplastics (PVC & TPO) and the necessity to have reliable in-situ performance data, future studies in the roofing domain should include higher percentages of these membrane materials.

#### 4.1.3 Process Automation

- The CAS performance metrics should be implemented in software, as data collection automation will reduce the number of transmission/transposition errors from paper-based systems.
- Automated data collection should use "wearable computers" running on well-accepted O/S platforms and stored in well-known database formats.
- Data collection for any portfolio should always use standardized tools.
- Need robust software interface to allow users to override the computer-generated priorities.
- Work collaboratively with IAI to develop IFCs in roofing domain as well as IFCs for generalized asset management.

#### 4.1.4 BELCAM Data Collection

- Continue data collection for upcoming years for the BELCAM project (NSERC 2001).
- More data need to be collected to get a full representative sample of the roofs in-situ and the roofs being constructed.
- Continue data collection to enhance the Markov Chain models and CBR approach (NSERC 2001).

#### 4.1.5 Service Life Models

- Develop generalized models for deterioration as temporary substitutes until sufficient data are available (Vanier 2001).

- Need to refine the risk model to take into account different sections of large buildings that have different occupancies and therefore different risks.
- Need to document protocols for multi-objective optimization (Kyle *et al* 2002a).

## 5 SUMMARY AND CONCLUSIONS

This section is broken into the six categories discussed in the previous two sections.

*Condition assessment survey protocols:* It can be assumed that condition assessment surveys (CAS) are representative of the current state of the roofing infrastructure for governmental and para-governmental agencies in Canada. The same holds true for the distresses; they are representative of both the type and the proportions found in the general population of these types of roofs. It can therefore be concluded that CAS is a viable method to determine the performance of in-situ roofing systems.

*Data Amalgamation:* Large organizations with considerable assets should standardize on methodologies of performance metrics, if not the software tools, for the data collection.

*Data Analysis:* More data are required to implement Markov Chain models in the various construction domains. This will only occur if there is a rapid increase in the use of computerized maintenance management models (CMMS) in industry and if there is continued and strong interest in CAS performance metrics.

*Service Life Prediction Modeling:* This is an essential component of any asset management system. Life cycle costs cannot be calculated without establishing a viable service life for each and every component. Markov Chain models are one of the most promising technologies to determine the remaining service life; however, techniques such as CBR are also reliable.

*Maintenance Prioritization:* Asset managers require decision-support tools that allow them to prioritize the plethora of maintenance and repair projects. To date, subjective techniques are predominant; however, in the future, techniques employing objective data such as condition, performance, risk of failure and life cycle costs will dominate.

*Decision Support Software:* There is a need for continued research in these areas. The BELCAM research will continue for an additional two years in collaboration with a number of Canadian universities (NSERC 2001); in course of this research additional roofing data will be collected and industry foundation classes (IFC) will be investigated as a potential model for data exchange.

The project identified and developed the essential constituents for comprehensive life cycle asset management. BELCAM achieved its two original objectives: (1) to develop techniques to predict the remaining service life of building envelope components and (2) to develop procedures to optimize maintenance management at a network level.

## 6 ACKNOWLEDGEMENTS

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# Comparative Study Of Two Concrete Bridges In Marine Environment

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**Summary: This paper presents the procedure used in assessment, evaluation, diagnosis and design of prestressed concrete bridge structure damaged by steel corrosion on the two bridges on the Adriatic coast. The bridges located in a marine environment have shown severe damages. Today, after 21 years in use, the Krk Bridge from mainland to the island of Krk begins having problems connected with steel corrosion in concrete, caused by chloride ions. That also appeared on the Vir Bridge after 18 years of use.**

**The damages have been investigated in three steps: visual prospecting detailed testing of representative locations and laboratory investigation of microstructure. The question of representative number of tests is discussed.**

**The environmental exposure varies for different parts of the structure and it results with the different degrees of damages This paper reports results of the chloride content in the concrete. The service life of the bridges is calculated based on the chloride content in the concrete. The experimental results were compared to the theoretical values of two different concrete qualities, high one and low one.**

**Keywords: chloride corrosion, marine environment, visual categorization, chloride ions distribution, corrosion propagation time**

## 1 INTRODUCTION

Chloride corrosion is one of the most common causes of the reinforced concrete deterioration in marine environment. Influence of the chloride penetration is recently studied on the two concrete bridges in the Adriatic Sea where considerable damages appeared. It was attempted to find relevant parameters for construction design in the future and to help in repair work already damaged constructions.

Vir Bridge was built at 1976 and the length of the bridge is 378 m (Loncaric 1988). The bridge has ten spans and the highest girder is 10 meter above the sea level. The Vir Bridge is founded on concrete piles, covered with reinforced concrete pile cap close to the sea level with a specified strength of 30 N/mm$^2$. The columns from the pile cap till the head beams are different heights from 3.0 to 10.0 m. Prestressed girders and three diaphragms that connect each pair of girders have a specified strength of concrete 40 N/mm$^2$. The main reason for occurrence of damages is reinforcement corrosion due to high chloride quantities penetrated into concrete. Wetting and drying cycles, temperature cycles as well as freezing and thawing cycles also accelerated the degradation.

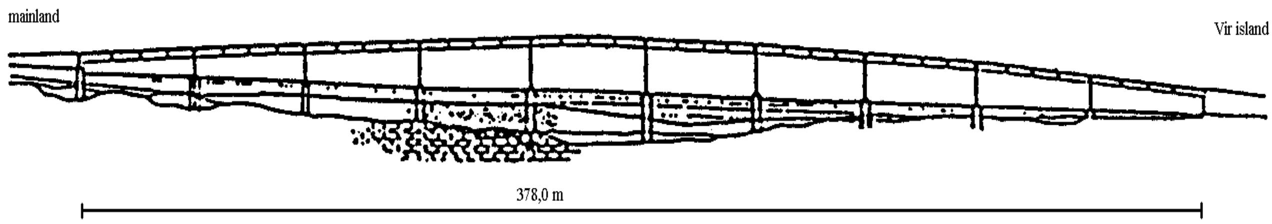


**Figure 1. Scheme of the Vir bridge**

Krk Bridge was built in the year 1980. In that time, that was the bridge with the largest span of the arch in the world (Sram 1981). Transition from the mainland to the island Krk was realized with two bridges: mainland - island St. Marko with the span

390 m and island St. Marko - island Krk with the span 244 m. Specified strength of the arch is 50 N/mm$^2$ and columns have a specified strength of concrete 35 N/mm$^2$. In the last few years, damages observed on the bridge bring to necessity of the investigations by means of which it could be find the most appropriate way for repair work. It was important to find the cover depth and to determine chloride content in the representative cross-sections.

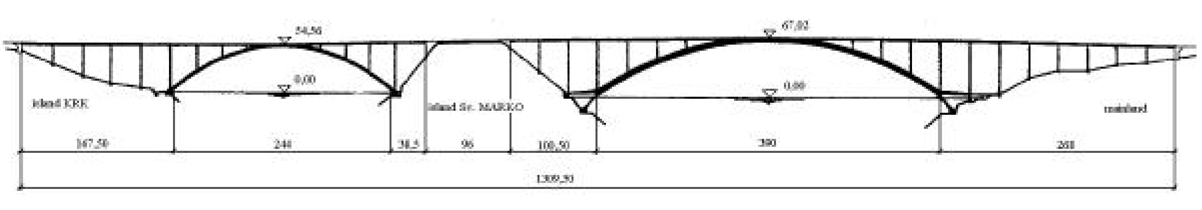


**Figure 2. Scheme of the Krk bridge**

## 2 EXPERIMENTAL WORK

Investigation was consisted of visual investigation with categorization on the basis of outward appearance and detailed in situ and laboratory tests of specimens taken from representative tests.

### 2.1 Visual investigation

Division of defects, damages and deterioration process were done according to following schedule:

I - surface irregularities

II - deeper defects

III - visible deterioration process

IV - deep damages

V - heavy damages

Categories I and II include air bubbles, surface irregularities, porous spots, leaching and cracks <0.1 mm. These failures are results of defects made during the construction. Category III presents discernible deterioration processes like exudation, popouts and scaling. The fourth and fifth category are damages and obvious consequences of advanced deterioration processes (Fig.3). For each structural element the percentages of certain categories is shown in Table 1. for Vir bridge and in Table 2. for Krk bridge.


**Figure 3. Vir Bridge - damage of the concrete pile**

**Table 1. Visual categorization (Vir Bridge)**

| *Structural element* | *Category share (%)* | | | | |
|---|---|---|---|---|---|
| | *I* | *II* | *III* | *IV* | *V* |
| PILES | 100 | 0 | 0 | 0 | 0 |
| PIERS AND PILE CAPS | 1 | 12 | 10 | 40 | 37 |
| GIRDER SUPPORTS | 5 | 18 | 14 | 34 | 29 |
| ABUTMENTS | 0 | 59 | 0 | 0 | 41 |
| PRESTRESSED GIRDERS | 37 | 40 | 11 | 5 | 7 |
| DIAPHRAGMS | 80 | 20 | 0 | 0 | 0 |
| DECK SLAB AND PATHS | 58 | 28 | 5 | 5 | 4 |

**Table 2. Visual categorization (Krk Bridge)**

| *Structural element* | *Category share (%)* | | | | |
|---|---|---|---|---|---|
| | *I* | *II* | *III* | *IV* | *V* |
| ARCH - BOTTOM SIDE | 0 | 82 | 4 | 7 | 7 |
| ARCH - NORTH SIDE | 0 | 73 | 18 | 8 | 1 |
| ARCH - SOUTH SIDE | 0 | 66 | 21 | 5 | 8 |
| DECK SLAB | 74 | 3 | 23 | 0 | 0 |
| PRESTRESSED GIRDERS | 0 | 87 | 9 | 3 | 1 |

### 2.2 Detailed testing

*Vir Bridge*

Twelve representative spots of categories I through III were selected for detailed inspection and verification of visually evaluated categories. These spots were tested by exact measurements of parameters defining their structure and properties. The categories IV and V represent obvious damages, therefore further investigations were not necessary.

Pull-off tensile strength results vary from 0.75 to 4.05 N/mm$^2$. That means that the skin of the concrete belonging to categories I to III had good strength. By Schmidt hammer was tested homogeneity of concrete. The average index is 51.3, minimum value is 41.2 and maximum value is 55.5 with standard deviation 3.6. It follows that the surface homogeneity is good. The cores were taken at 11 spots that are chosen after testing the hardness index. The strengths of cores were in range from 38.4 to 52.7 N/mm$^2$. The characteristic strength of concrete has been evaluate as 10% fractile, $f_{ch}$=42.2 N/mm$^2$.

Possible defects or damages around the prestressing tendons were trying to detect by ultrasonic puls velocities in concrete. The pulse velocity measurements have shown inhomogenities inside the girders (nests and holes). During the visual investigations of presstresed girders the cracks caused by swelling of corroded reinforcement were registered mainly close to their supports. The concrete cover depth was varied from 0.5 cm (very thin) to more than 6 cm. In areas evaluated as category V, the concrete cover depth is spalled due to reinforcement corrosion.

According to the standard test method ASTM C 876 the measurements of half cell potential were done. The measurements shown changes in the range of -233 mV to -644 mV, showing high risk of steel corrosion process. Capillary suction is very intensive for all specimens. Obviously, the high salinity of concrete contributes to the rapid suction of water into the concrete.

The state of the concrete was also investigated on thin slices cut from the surface area and from the depth of cores by X-ray diffraction, microscopically and by chemical gravimetric methods. Specimens taken close to the surface were with plenty cracks, spreading irregularly through cement paste and aggregate grains. Some microcracks result from corrosive processes. On the specimens cut from the deeper part of the concrete, the damages are similar to the above ones, but in smaller amount. By X-ray diffraction applied to the surface zone $CaCO_3$ was identified, and for the inner part of concrete $CaCO_3$ and $Ca(OH)_2$ were observed.

Chloride contents in concrete were tested from fourteen characteristic locations for samples taken from four subsequent depths (0-1 cm, 1-2 cm, 2-4 cm and 4-6 cm). The results are plotted in diagrams, and two examples are shown in Fig. 4. They were divided into groups characterized with similar microclimatic conditions.

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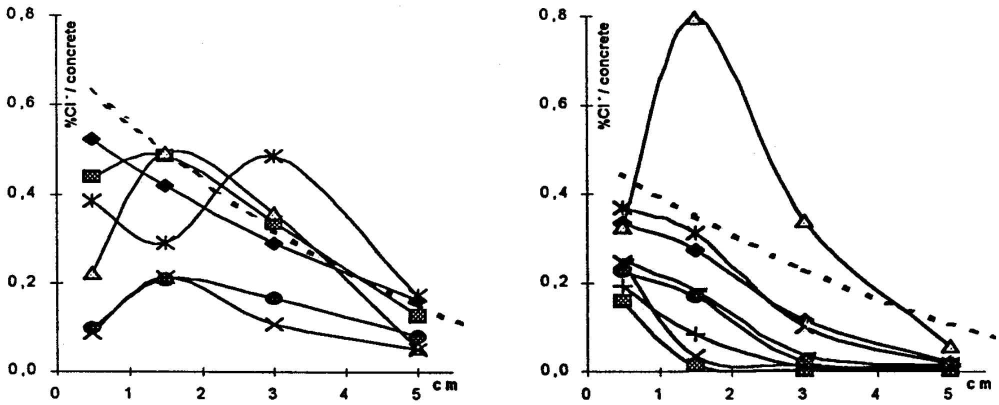


a) b)

**Figure 4.Chloride ions distribution a) Outside of girders from north; b) Inside the girders. The dashed line is theoretical distribution. (Vir Bridge)**

*Krk Bridge*

By Schmidt hammer was tested homogeneity of concrete. The average index is 57.4, minimum value is 52.4 and maximum value is 62.7 with standard deviation 2.3. The cores were taken at representative spots and the characteristic strength of concrete has been evaluate as 10% fractile, $f_{ch}$=70.2 N/mm$^2$.

The most considerable failures are observed on the longitudinal connection of the prefabricated elements of the arch. It is visible the spalling of the concrete (Fig.5) and the cross section of the reinforcement is decreased.

**Figure 5. Krk Bridge - damage of the concrete column**

The measurements of the half-cell potential on the undamaged surface showed the small possibility of corrosion occurrence. This is probably the result of the high concrete quality. However, on the spot with smaller concrete cover depth, corrosion occurrence is probable.

## 3 RESULTS ANALYSIS

The rate of chloride ions penetration into concrete depends on the concentration of chlorides in the surface layer, climatic circumstances (humidity and temperature) and concrete properties. The chloride distribution can be conveniently approximated by parabola fulfilling the second Fick's law of diffusion:

$$C_x = C_s \left( 1 - \frac{x}{2\sqrt{3 \cdot D_{Cl^-} \cdot t}} \right)^2$$

where:

$C_x$ - concentration of chloride ions in the depth x

$C_s$ - concentration of chloride ions at the surface

$D_{Cl^-}$ - diffusion coefficient

t - time

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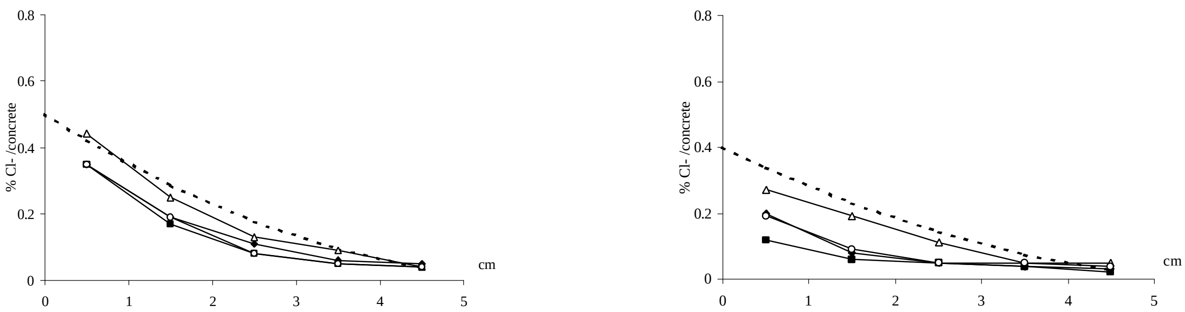
Majority standards define critical chloride ions content at the reinforcement as 0.4% cement content for reinforced concrete and 0.2% for prestressed concrete. The theoretical diffusion coefficient is $D=10^{-8}$ $cm^2/s$ for specified strength 40 $N/mm^2$ and $D=10^{-7}$ $cm^2/s$ for 30 $N/mm^2$.

*Vir Bridge*

Figure 4. shows measured data in comparison with theoretical profile for $C_s$ = 0.7 and 0.5%. It should be mentioned that for the sake of theoretical calculations in design of structures exposed to marine environment, several authors recommend $C_s$=0.3% to 0.4% (RILEM Report 14 1996). The higher $C_s$ results probably from the higher concrete porosity. The concentration of chlorides on the outside surface exposed to northern winds is significantly higher. On that side the chloride ions content exceeds the standard permissible maximum even at 6 cm depths and more. In majority cross-sections, the maximum is located at depths between 1 and 2 cm. That is the result of periodic washing out of salts by rain. The girder on southern and inner side have almost half the chloride content compared to northern side. The exception is the highest chloride content measured in the girder closest to the sea level in the splashing zone. The effect of splashing and washing out here is the most visible. The structural parts placed under the sea water level have significantly lower level of chloride ions.

*Krk Bridge*

Comparison of measured data and theoretical distribution of the chloride ions is shown on the Fig. 6. For the theoretical distribution it is chosen the theoretical diffusion coefficient $D = 10^{-9}$ $cm^2/s$ and age of the concrete was 20 years. Results were observed according to different heights and orientation. The characteristic cover depth of the concrete has been define as 10% fractile, c = 3.5 cm. The concentration of chloride ions at the surface, $C_s$ was determined to fit the measured data at the depth 3.5 cm.




a) b)

**Figure 6. Krk bridge (column) - chloride ions distributions: a) 15 m above the sea level; b) 30 m above the sea level**

## 4 COMPARISON OF THEORETICAL AND EXPERIMENTAL VALUES

The corrosion propagation time is:

$$t_1 = \frac{\Delta R_{max}}{r}$$

where:

$\Delta R_{max}$ - the greatest loss of steel bar diameter

r - the rate of corrosion

The chloride concentration in piers at the splashing zone was high enough to start steel corrosion at the very beginning ($t_0 = 0$), The diameters of the main reinforcement were 20 and 22 mm and for stirrups 8 mm.

The time elapsed till the spalling of the concrete cover is:

$$t_1 = 80\frac{C}{Dr}$$

where:

C - the depth of concrete cover

D - diameter of steel bar

r - the rate of corrosion

The rate of steel corrosion depends on environment conditions and the most influencing parameter are humidity and temperature. The rate of corrosion can be expressed by the:

$$r = c_T \cdot r_0$$

where (Tuutti 1982):

$c_T$ - parameter showing the influence of the temperature,

$r_0$ - the rate of corrosion at 20°C, obtained by long term observations in certain climatic areas.

It follows:

$$t_1 = 80 \frac{C}{8 \cdot 0.73 \cdot 9} = 1.522 \cdot C$$

$c_T$ = 0.73 for Madrid, Spain (similar temperature conditions like on Adriatic coast),

$r_0$ = 9 µm/year for 50% RH (Alonso & Andrade 1993).

*Vir Bridge*

The theoretical values correspond well with visual categorization and as measured depth of concrete cover in the zones of higher chloride ions contaminated concrete. The quantity of chloride ions for all concrete cover depths measured on the bridge is above the critical quantity. The spalling of concrete cover on prestressed girders has happened in areas where the concrete cover is less than 20 mm. That corresponds well to the calculated values. The spalling of concrete cover is on piers reached 50 mm.

*Krk Bridge*

Three cases are interesting on the Krk bridge:

- the spalling of the concrete happened in the cross sections with thin concrete cover
- according to the concentration of the chloride ions in some cross sections, beginning of the reinforcement corrosion is possible
- appearance of the pitting corrosion is also possible and that is dangerous for the construction, because of the rapid development

**Table 3. Corrosion initiation time (Krk bridge)**

| *Structural element* | *height above the sea level* (m) | $C_s$ (% /concrete) | *Corrosion initiation time* (years) |
|---|---|---|---|
| ARCH - BOTTOM SIDE | 0-15 | 0,3-0,45 | 20,4-28,7 |
| | 15-50 | 0,25 | 35,3 |
| ARCH - NORTH SIDE | 5 | 0,65 | 16,4 |
| | 15 | 0,4 | 22,2 |
| | 15-50 | 0,4 | 22,2 |
| ARCH - SOUTH SIDE | 5 | 0,6 | 17,6 |
| | 15 | 0,5 | 19,0 |
| | 15-50 | 0,3 | 28,7 |
| COLUMNS | 5 | 0,45-1,2 | 12,8-20,4 |
| | 15 | 0,50-0,55 | 17,9-19,0 |
| | 30 | 0,4-0,8 | 14,9-22,2 |
| | 50 | 0,25-0,55 | 17,9-35,3 |
| PRESTRESSED GIRDERS AND DECK SLABS | 50 | 0,15-0,25 | 35,3-90,0 |

It can be seen that the process of reinforcement corrosion on the most spots already began. Concentration of the chloride ions depends on the height above the sea level and location of the structural element to north or south side.

## 5 CONCLUSIONS

Detailed investigation of representative locations on the Vir bridge has shown that damages classified as category I and II "hide" severe damages and deterioration processes that were not possible to detect during the visual inspection. By microscopic investigation plenty of microcracks and the deterioration processes of cement paste has been observed. The consequence is higher saturation of concrete with sea salts and high water absorption. Higher quantities of chloride ions in the depth of the concrete and half cell potential measurements suggest that steel corrosion is intensive although during the visual investigations these areas were classified into category I and II. It is concluded that the areas evaluated as categories I and II should be added to the category III.

The chloride ions concentration had the main influence on the rapid reinforcement corrosion. The presstresing tendons are not threatened. Two processes were evaluated:

- the decrease of the reinforcement diameter is significant on all the piers, pile caps and some places on the girders
- concrete is partly spalled on the places with thinner concrete cover; it raises the question when it is going to start spalling at areas with thicker concrete cover or in areas with better concrete quality or concrete less contaminated with chloride ions

Results of the investigation on the Krk bridge show that the great influence of the chloride ions content in concrete have the microclimate conditions (height above the sea level, exposition to severe northern wind and strong southern wind, the salinity of water).

Greater damages are on the Vir bridge due to less specific strength and low quality of the concrete than on the Krk bridge.

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# An Assessment Tool For The Durability Adaptability And Energy Conservation Of Buildings

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**Summary: One of the main outcomes of our research project on sustainability of buildings is an assessment tool for the durability, adaptability and energy conservation of buildings (DAEC Tool) which is now available as software. The DAEC Tool has been tested on existing higher education buildings and a new building design for community hospital. This paper describes the development of the DAEC Tool and its envisaged application in developing a design brief, building design, evaluation of design options, and assessment of existing buildings.**

**Keywords: adaptability, assessment tool, durability, energy conservation**

## 1 INTRODUCTION

The UK construction industry aims to contribute to sustainable development by adopting new policies and practices which have a more positive impact on economic, social and environmental systems (DETR 2000). Improvements are sought at all stages of the construction process, such as the land use, building design, construction process and building management. Our research aims to contribute to building design and management by examining the issues related to the durability, adaptability and energy conservation of buildings. The objectives of the research project were:

- to develop the metrics for durability, adaptability and energy conservation (incorporated into the DAEC Tool),
- to test the metrics on existing buildings and new building designs,
- to identify conflicts between the examined qualities and
- to propose a strategy for their reconciliation.

The environmental impact of buildings has been a focus of research efforts in the UK (Baldwin *et al* 1998; Howard *et al* 1998) and internationally (Cole and Larsson 1999; Anink *et al* 1996), mainly through the assessment of environmental impact of the energy and materials used throughout a building life. However, other building features affect their sustainability. In the context of sustainable development, the design of long life adaptable structures is considered as one of the main strategic goals in the construction industry (Kohler 1999). This has also been acknowledged in the latest version of the Canadian Green Building environmental assessment tool (GB Tool) by including longevity in its scope (Cole 1999). The design life of a structure is dependent on more than resisting environmental and non-environmental agencies; if it is adaptable, it is likely to have a longer life (Burns 1992; Chapman 1992). It is argued that the public interest requires high quality long-life construction which is 'conservant' with respect to energy and materials (Chapman 1992). This can be achieved through quality, durability, maintainability, and the ability to adapt, dismantle, relocate, or recycle (Chapman 1992).

Bernstein's (1999) definition of 'sustainable' buildings comes close to the definition of adaptable buildings:

> 'As industry and academic leaders discussed during the CERF/CIB Symposium, 'sustainable' means being able to meet the multiple requirements of society through the life cycle of a building or structure'.

If a building is not suited to changing circumstances, it is vulnerable to becoming poorly utilized, prematurely obsolete and unable to accommodate changing technology that could make the building better able to meet contemporary user needs, as well as increasing the efficiency of building systems (Grammenos and Russel 1997). Obsolete buildings are evidence of wasted natural resources and inadequate use of land, and this may be avoided if buildings are adaptable. While traditional building design has been based on the ergonomic requirements of an average, healthy adult, ignoring other human conditions, more recent architectural design aims towards '*differentiation*, not uniformity in designing the built environment' (Preiser *et al* 1991). Building design needs to consider the entire human lifespan, including special needs. It is also argued that the universal design approach – simply designing all products, buildings and exterior spaces to be usable by all people to the greatest extent possible – is a sensible and economical way to reconcile the integrity of a design with human needs in the environment (Mace *et al* 1991).

The fact that about 56% of the energy consumed, both nationally and internationally, is used in buildings (Harvey and Ashworth 1996), shows the opportunities and responsibilities for designing energy efficient buildings. In the construction industry energy is used for the extraction and manufacture of building materials and components, their transportation to the building site, the construction process, the running of the building, maintenance, adaptations, deconstruction and disposal. Energy conservation of buildings pertains to all these phases of building life.

The complexity of factors influencing sustainability of the building industry demands a holistic approach in analysis, assessment and development of policies. Research efforts focus on two main areas: the development of an integrated sustainability assessment method, and the deepening of knowledge about individual factors that affect the sustainability of buildings. With regard to the specific building design features which affect the sustainability of buildings, it is necessary to examine whether the improvement of some building features might have an adverse affect on others. Therefore, the following points need to be examined:

- the relationship between the factors that influence the sustainability of buildings such as the durability, adaptability and energy conservation,
- possible conflicts between them, and
- strategies for their reconciliation.

## 2 DEVELOPMENT AND APPLICATION OF ASSESSMENT TOOL ON EXISTING BUILDINGS

The research programme included the testing of the DAEC Tool on existing buildings which were selected from the John Anderson Campus at the University of Strathclyde. The selection process entailed examining all the campus buildings in order to identify those which were most suitable for this research. The aim of the selection process was to identify buildings with different use, date of completion, type of construction, design brief (purpose built or adapted) and HVAC systems. With regard to the use, the selected buildings fall into two main types: educational (teaching facilities, labs, library) and residential (student residences), but there are also two buildings with mixed use (teaching and accommodation). The dates of building completion range from 1905 to 1998. The types of construction include those used at the beginning of the 20$^{th}$ century, e.g. heavy brick walls, and those developed later on, e.g. concrete and steel frames. In total 11 buildings were selected, of which two were not purpose built but adapted for higher education use. The HVAC systems vary from a number of different systems used within one building; warm water gas-powered central heating, electrical heating; to a passive solar system combined with backup heating and heat recovery system. The case studies about selected buildings, plans of the buildings and their adaptations, and visits to the buildings enabled the assessors to obtain a comprehensive understanding of the buildings in order to use the DAEC Tool effectively on the buildings which they assessed. The DAEC Tool was tested at three workshops. After the first workshop the final tabular format of the assessment forms was defined (Tables 1, 2 and 3).

**Table 1. A segment of the durability assessment table with reduced size of table cells**

| Assessment of design elements | | | | | | | | | Life cycle cost | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | | | | | 5 | 6 | 7 | 8 | 9 | 10 |
| Building elements | Design characteristics | Required service life | Assessment scale | | | | | AV | IC | CC | CD | CR | PV |
| FOUNDATIONS | | | 1 | 2 | 3 | 4 | 5 | | | | | | |
| Type 1 | Strength | | | | | | | | | | | | |
| | Ph/ch properties | | | | | | | | | | | | |
| Type2 | Strength | | | | | | | | | | | | |
| | Ph/ch properties | | | | | | | | | | | | |

AV – average; IC – initial cost; CC – cleaning cost per year; CD - cyclical decoration cost; CR – cyclical replacement cost; PV – present value; Ph/ch – physical/chemical

**Table 2. A segment of the adaptability assessment table**

| Assessment of building design | | | | | | | | Costs | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | | | | | 5 | 6 |
| Elements | Assessment criteria | Level of adaptability | Assessment scale | | | | | Initial cost | Adaptation Cost |
| | | | 1 | 2 | 3 | 4 | 5 | | |
| SITE: | | | | | | | | | |
| Possibility of expansion | Expansion possible in all directions: around the building and upwards. | High | | | | | | | |
| | Expansion possible either around the building or upwards. | Medium | | | | | | | |
| | Expansion not possible. | Low | | | | | | | |

**Table 3. A segment of the energy conservation assessment table**

| 1 | 2 | 3 | | | | | 4 |
|---|---|---|---|---|---|---|---|
| Design features | Energy conservation criteria | Assessment scale | | | | | Costs |
| | | 1 | 2 | 3 | 4 | 5 | |
| Building orientation | All facades exposed to natural light. | | | | | | |
| | Two facades exposed to natural light. | | | | | | |
| | Only one facade exposed to natural light. | | | | | | |

### 2.1 Assessment method

An objective assessment method is used in the DAEC Tool wherever the performance targets are agreed and are available (e.g. service life in durability assessment; U values, energy consumption etc. in energy conservation assessment). However, there are no performance targets for adaptability of buildings, and a subjective assessment method had to be applied in the adaptability assessment. The adaptability criteria were based on published research and on judgements of experts regarding the design of adaptable buildings.

Scoring system awards 1 point to the lowest performance and 5 points for the best one. Since the number of the examined building design elements, features or performance targets is different in each assessment table (19 in the durability assessment, 28 in the adaptability assessment and 24 in the energy conservation assessment) the maximum number of points in each assessment is different (95 in the durability assessment, 140 in the adaptability assessment and 120 in the energy conservation assessment). In order to compare the aggregated results, they are presented as percentage of the highest possible score in each assessment.

Weighting is not included in the DAEC Tool for the following reasons:

- When assessing building design achievement goals, e.g. durability, adaptability and energy conservation, it is difficult and often unhelpful to determine which one is more or less important. For example, higher education buildings house activities that are likely to exist for a long period of time. Thus, it is beneficial if they are durable and easily maintained. Equally, it is beneficial if their design contributes to energy conservation and reduces the environmental impact of energy-in-use. Since the curriculum in higher education institutions frequently changes to meet the developments in education, the buildings need to be adaptable. It can be concluded that the durability, adaptability and energy conservation in higher education buildings are equally important. This may also be the case with some other types of buildings.
- When assessing building elements and features, the importance of their impact on achievement goals is related to the performance targets of building elements and systems. For example, if the required performance for building durability is 200 years, the service life of all its elements, components and systems does not necessarily need to be the

same, and it will be determined with regard to planned replacements and whole life costs. The required service life of the building elements and systems is determined by clients and designers and provided in the assessment tool as design brief requirements. Since it is expected that clients and designers will require the service life of foundations and structure to be equal to the planned building life, this does not need to be emphasized by weighting.

- Weighting of adaptability features is related to building type and planned adaptations and cannot be determined in absolute terms. For example, adaptations can be planned as minor changes within the same use, significant changes for similar use or radical changes for completely different use. It can be expected that the different design features would be considered as the most important in each of these cases, e.g. the possibility of extension would not be considered important for minor changes within the same use, but it would be very important for complete change of use.
- Concerning the weighting of energy conservation features and their environmental impact, there is not yet general consensus on the weighting system (Cole 1999). Some assessment methods implicitly include weighting, e.g. BREEAM (BRE 1990), but no systematic basis for weighting is described (Levin 1997). Weighting implies prioritising environmental problems on a scientific basis, but the complexity of the task has not been satisfactorily addressed yet (Levin 1997).

### 2.2 Durability metrics

Durability is defined as service life, i.e. actual period during which no unacceptable expenditure on maintenance or repair is required. The assessment table (Table 1) lists the following building design elements and systems in order to examine their performance profile against the required performance profile: foundations, structure, roof and covering, building envelope, partitions, floor finishes, ceiling finishes, wall finishes, stairs, windows, doors internal and external, fittings, HVAC system, lighting, water plumbing, sewage system, lifts. Short description of each element is included. Should two or more different specifications exist for one element (ex. two types of structure or building envelope, etc.), each one is presented since their life span may not be equal. As appropriate, the following characteristics are assessed: strength, physical/chemical characteristics, fire resistance, ease of maintenance and appearance. If there is more than one specification for the same building element/system, e.g. two types of structure, finishes etc., all are assessed and the average assessment result is calculated.

### 2.3 Adaptability metrics

Adaptability is defined as:

- Ease of change of building spatial organisation within the same use
- Ease of change for new use
- Ease of change of technology and services
- Ease of use for people with different physical abilities.

Assessed design features are spatial, structural and services design features, and design features that affect ease of use of the spaces by occupants with different physical abilities. The following design features are assessed (Table 2): site (possibility of expansion, access for pedestrians, access for services), interior layout and design (completeness of brief, flexibility of layout, grouping of functions, average main room size, provisions for disabled), structure (strength of columns/walls, column density/span, floor-to- ceiling height, floor loading, floor structure, removability of partitions), HVAC system (plant location, plant size space wise, access for people, access for equipment, ducting access), electricity (extra load, wiring space, access for servicing), water (supply, capacity), sewage (capacity), drainage (capacity), and lifts (capacity, extra space).

Adaptability criteria define how the following levels of adaptability can be achieved:

- Low adaptability – design features are appropriate for minor changes within the same use (e.g. organisational)
- Medium adaptability – design features are appropriate for more complex changes within same use (e.g. technological) and for similar use (e.g. from student residences into a hotel)
- High adaptability – design features are appropriate for complete change of use (e.g. industrial building into a library).

### 2.4 Energy conservation metrics

Energy conservation of buildings aims to reduce the energy used during the whole life cycle of buildings (production of building materials and components, their transportation to building site, construction process, running of building, maintenance, adaptations, deconstruction and disposal). Energy conservation criteria examine the following building design features that affect energy conservation and performance targets related to energy consumption and the environmental impact of energy-in-use (Table 3): building orientation, exposure to winds, overshadowing by neighbours, building form, U values, building plan vs. heating adjustment, type of glazing, solar energy use and control (overheating, glare, solar energy use and energy production), cooling/ventilation system, lighting system, lighting control, plan depth, day lighting area, resources for lighting, resources for heating, $CO_2$ emissions, $NO_x$ emissions, ozone depletion, recycling of energy and materials, embodied energy, energy-in-use consumption, and energy-in-use monitoring.

Energy conservation criteria define how the following levels of energy conservation may be achieved:

- Low energy conservation – design features and performance targets which just or do not meet present standards or average practice
- Medium energy conservation – design features and performance targets which are better than present standards and are considered good practice
- High energy conservation – design features and performance targets which significantly exceed present standards and are considered the best practice.

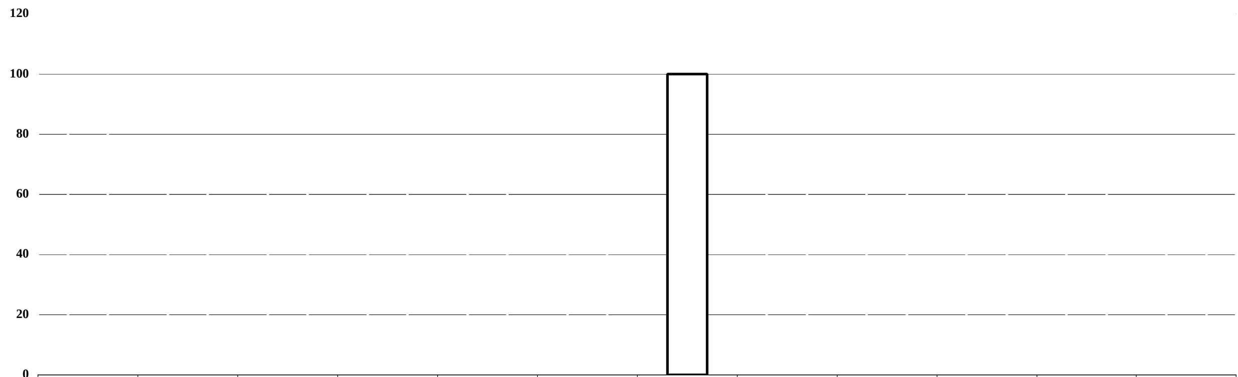
Since the types of design features and targets for energy conservation may change due to innovations in building design, better efficiency of services, or increased use of renewable energy, the criteria may be modified. The proposed criteria are thus related to the current state-of-art in building design, current performance targets, and to building type.

### 2.5 Financial considerations

In the table for durability assessment there is a provision for input of whole life costs (Table 1, columns 6 to 10). The table for adaptability assessment includes a provision for input of initial costs and the costs which would occur after the building has been built in order to achieve the level of adaptability which would have been provided by the initial investment (Table 2, column 5 and 6). The table for energy conservation assessment includes a provision for input of costs for all specifications and building features which contribute to energy conservation (Table 3, column 4).

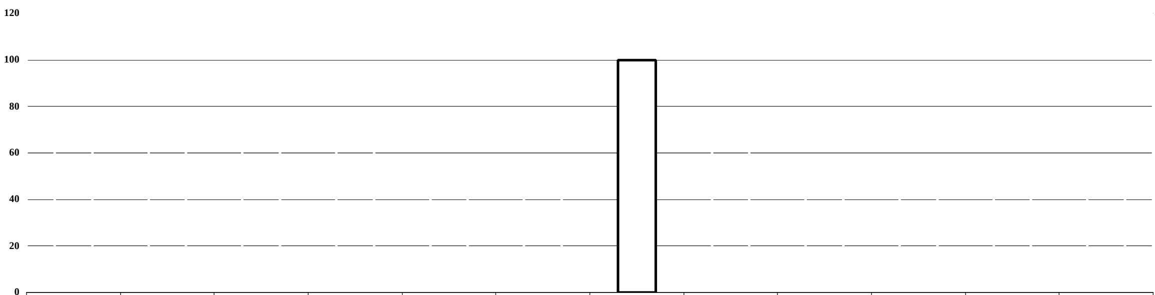
### 2.6 Comparative assessment of selected buildings

The final assessment of the selected buildings provides a comparative overview of their durability (Fig. 1), adaptability (Fig. 2) and energy conservation (Fig. 3). This overview facilitates the decision making process about priorities in building maintenance, about their suitability for adaptations, and about the measures which will improve the energy conservation. The application of the DAEC Tool shows that the assessment results correspond to the qualitative analysis of the selected buildings. This means that the building design elements and features listed, and the proposed criteria in the DAEC Tool provide an adequate broad brush tool for assessment of durability, adaptability and energy conservation of existing buildings.




| | SGBS | H.SC | R. CLLGE | CURRN | L.TWR | W. DNCN | MAX | GOOLD | CHLLRS | JB&TC | MRRAY | BK BECK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Series1 | 85 | 75 | 73 | 66 | 64 | 57 | 100 | 79 | 67 | 63 | 63 | 56 |

**Figure 1. Durability assessment results of selected named teaching facilities (left group) and student residences (right group) presented as a percentage of the highest possible score (Max)**




| | H.SC | SGBS | CURRN | R. CLLGE | L.TWR | W. DNCN | MAX | GOOLD | CHLLRS | JB&TC | MRRAY | BK BECK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Series1 | 74 | 70 | 64 | 61 | 58 | 58 | 100 | 72 | 57 | 55 | 54 | 51 |

**Figure 2. Adaptability assessment results of selected named teaching facilities (left group) and student residences (right group) presented as a percentage of the highest possible score (Max)**

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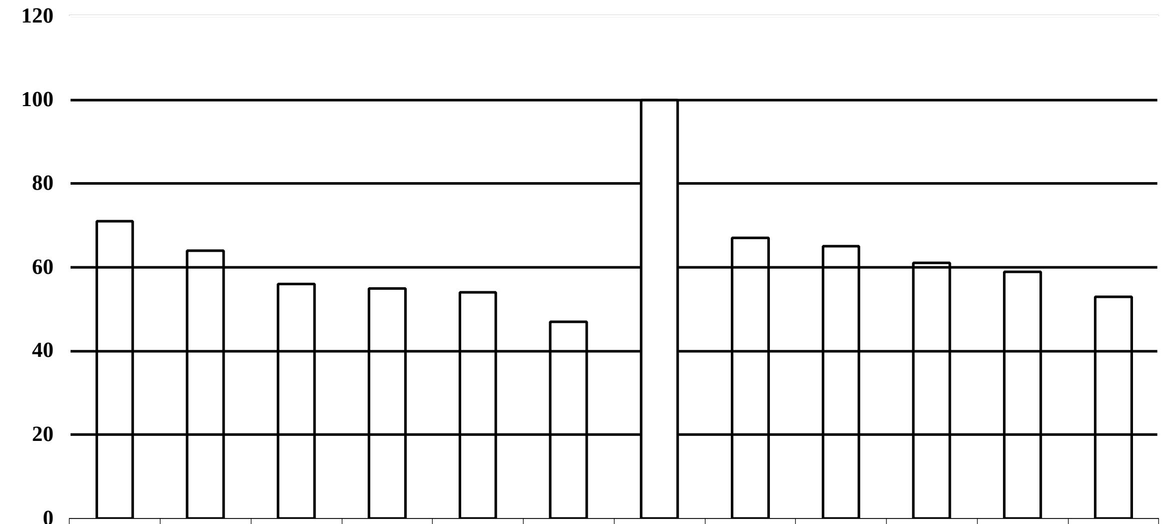
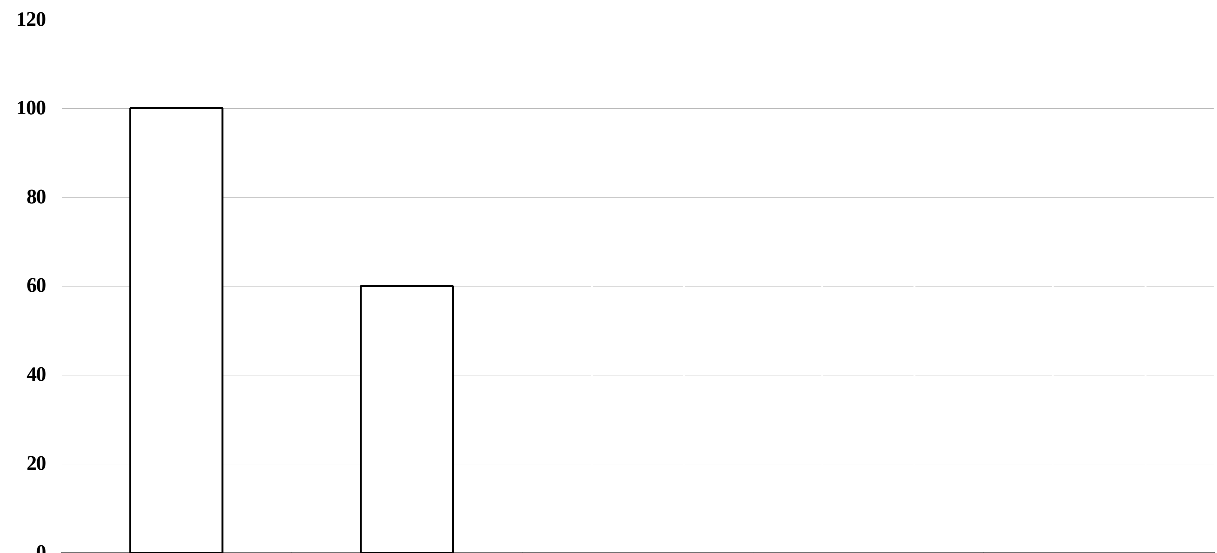



| | H.SC | SGBS | R. CLLGE | W. DNCN | L.TWR | CURRN | MAX | JB&TC | GOOLD | CHLLR S | MRRAY | BK BECK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Series1 | 71 | 64 | 56 | 55 | 54 | 47 | 100 | 67 | 65 | 61 | 59 | 53 |

**Figure 3. Energy conservation assessment results of selected named teaching facilities (left group) and student residences (right group) presented as a percentage of the highest possible score (Max)**

## 3 APPLICATION OF THE DAEC TOOL ON A NEW BUILDING DESIGN

HBG Construction Ltd was involved in the development of the DAEC Tool during the initial consultation process and at the first workshop when it was tested on existing buildings. The DAEC Tool was used in the preliminary assessment of their new building design for a hospital project (Fig. 4). The client's requirement for a 25-year service life of a building is represented by the second column in Fig. 4, as 60% of the highest possible score because the scoring system awards 3 points to the building elements which meet a required service life. The durability assessment shows that the proposed building design exceeds the client’s requirement. The adaptability assessment indicates that the building design allows easy adaptation within the same and similar uses, especially regarding possible extensions, changes and increase of the services capacity. The assessment of the energy conservation / environmental impact of energy shows that the building design provides many features which contribute to energy conservation and low environmental impact of energy-in-use.




| | MAX | 25 year service life | Durability | Adaptability | Energy conservation |
|---|---|---|---|---|---|
| % | 100 | 60 | 71 | 77 | 72 |

**Figure 4. Assessment results of a new building design for community hospital**

The assessment results of all three qualities fall within a close range (71%/72 %/77 % of the highest possible score) reflecting the balance between them. This balance is important because it indicates that the pursuit of any one of the building design achievement goals has not adversely affected others. There are no similar assessments of other hospital buildings for reference and comparison of the assessment results. However, the assessment scale from 1-5 defines five levels of quality, where the lowest gets 20% and the best 100% of the highest possible score. The assessment results show that the building design falls within the second highest group (60-80% of the highest possible score).

The assessment enables the client to have a better insight into the proposed building design and to decide where, if needed, changes may be required. The transparency of the assessment method provides the possibility for a review of the assessment and a dialogue between the client and the designers regarding building design features, possible modifications and cost implications of different specifications.

Application of the DAEC Tool to other types of buildings is possible because the evaluation tool has been designed as a framework which allows for modifications of the following elements:

- *Types of building design elements and features which are assessed*, by amending the list of the assessed elements and features.
- *The assessment criteria*, by defining the criteria which are the most appropriate for the building type. It can be expected that the durability, adaptability and energy conservation criteria, for example, for industrial buildings will vary from those of residential buildings.
- *Achievement goals*, by defining durability, adaptability and energy conservation benchmarks according to the most recent targets.
- *Scoring system*, by selecting a different type of scoring system.

## 4 IDENTIFICATION OF CONFLICTS IN BUILDING DESIGN

Identification of conflicts between the achievement goals of building design is an analytical process which has relied on different methods in building design practice. Broadebent (1988) points out that the ‘new maths’, with a certain amount of statistics, has been almost as influential in the development of new design methods as all other sources and disciplines together. Fascination with new analytical tools has sometimes led to abstract concepts which seem detached from their end-purpose, design for real people (Broadabent 1988). In systematic design, characterised by examining one or more partial solutions to each project requirement, the solutions to some project requirements will conflict with the solutions to others (Jones 1966).

Conflicts between building features/specifications which affect durability, adaptability and energy conservation were identified by adapting Luckman’s technique named AIDA – Analysis of Inter-connected Decision Areas, for graphic presentation of conflicts in building design (Luckman 1969). The technique is based on the fact that a design problem can be expressed in terms of various factors, and that for most of these factors several solutions are possible. A particular solution which is selected for one factor will encourage certain solutions for other factors and inhibit others. Where the choice of a particular option in one decision area will inhibit the adoption of a certain option in another decision area, then the two are linked – representing their incompatibility.

Luckman’s technique is used to identify the conflicts in an existing building or a building design by linking the conflicting design features/ specifications/ environmental impact parameters listed in the DAEC Tool. Conflicts identified in the original and adapted forms of the Royal College building are presented in this paper as an example of the method. The links representing the incompatibilities are numbered, the conflicts are described and proposals for their reconciliation provided (Table 5).

An analysis of conflicts in the building design of selected buildings shows that 46% represent conflicts between durability and adaptability design features, 41% between durability and energy conservation, 8% between adaptability and energy conservation, and 5% arise as a result of combination of features related to durability, daptability and energy conservation. The highest number of conflicts was identified in the Royal College building which is the oldest, largest and most complex of all the examined buildings. The number of conflicts is higher in the teaching facilities than in the student residences. Teaching facilities house different uses (e.g. laboratories, lecture theatres, offices, catering, libraries, computer rooms etc.) which have different spatial and environmental requirements, while student residences consist of multiple flat units.

**Table 5. Description of 18 identified conflicts in the Royal College building (built in 1905 and adapted many times) and proposals for their reconciliation**

| No | Link | *Description of the conflict* | Proposal for reconciliation |
|---|---|---|---|
| **1** | **D-A** | Roof type (complex) vs. adaptability (difficult access) | Accessible roof; roof strength which allows new loads; roof shape which lends itself to adaptation |
| **2** | **D-A** | Solid (brick) partitions vs. adaptability (difficult to remove) | Partitions which can be dismantled and re-used |
| **3** | **D-A** | Type of heating system (‘plenum’, centralised) vs. adaptability (inflexible) | Flexible, decentralised heating system |
| **4** | **D-A** | Ducting of heating (ducts and shafts built into brick walls) vs. adaptability (difficult access, obsolete after a new heating system has been installed) | Ducting which is accessible and could be easily dismounted |
| **5** | **D-A** | Electrical installations (type and low capacity) vs. adaptability (need for higher capacity) | Better quality and higher capacity of electrical installations |
| **6** | **D-A** | Wiring (type and location) vs. adaptability (lack of wiring space) | Ample wiring space |
| **7** | **D-A** | Lifts (type and capacity) vs. adaptability (not accessible for disabled, low capacity) | Large lifts, accessible for disabled |
| **8** | **D-A** | Type of building envelope & partitions (heavy, solid) vs. need for extension (e.g. for new lifts) | Building envelope and partitions which can be dismounted partially or completely |
| **9** | **D-A-EC** | Type of heating system (ducting, location of openings into rooms) vs. adaptability (flexibility of layout) and vs. local heating adjustments | Flexible heating system which allows dismounting the heaters, adding new ones, and local adjustment of heating |
| **10** | **A-D-EC** | Average main room size (large) vs. type of heating (inflexible), and vs. need for flexibility and local heating adjustments | “ |
| **11** | **D-EC** | Type of windows (still original, single glazed) vs. U values | Double glazing, or other types of glazing with low U values |
| **12** | **D-EC** | Type of windows (very large, no shading) vs. protection from overheating | Shading against overheating |
| **13** | **D-EC** | Means of heat delivery (openings near the floor and ceiling) vs. local heating adjustment | Heating adjustment which can be easily operated by occupants |
| **14** | **D-EC** | Type of heating system and resources for heating (originally coal, now gas, oil and electricity) vs. depletion of resources | Reducing the use of non-renewable resources for heating, and adopting reneawable energy resources |
| **15** | **D-EC** | Type of heating system vs. $CO_2$ emissions | “ |
| **16** | **D-EC** | Type of heating system vs. ozone depletion | “ |
| **17** | **D-EC** | Type of heating system vs. energy monitoring (no means for measuring energy consumption) | Decentralised heating, energy consumption meters |
| **18** | **D (A)-EC** | Variety and decentralisation of new energy systems vs. energy monitoring (centralised) | Separate metering of energy used for heating, lighting, and equipment |

D - durability features; A - adaptability features; EC – energy conservation features

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## 5 ENVISAGED APPLICATION OF THE DAEC TOOL

It is envisaged that the DAEC Tool will be used in the following stages of building design and building management:

- *Development of the design brief and identification of achievement goals.* The client may use the proposed assessment criteria for durability, adaptability and energy conservation to determine quality targets which need to be specified in the brief.
- *Building design*. When the achievement goals and quality targets have been built into the DAEC Tool, the design team can use the DAEC Tool for the assessment of different design options.
- *Client evaluation of building designs*. The evaluation tool can be used when a client needs to assess the projects submitted by different design teams. The assessors can use the evaluation tool as an aid in determining to what extent the projects meet the predefined quality targets.
- *Comparison of the whole life costs of different options for achieving the quality targets.* The evaluation tool enables comparison between the desired quality targets and the costs for their achievement.
- *Assessment of durability, adaptability and energy conservation of existing buildings*. The need for evaluation of existing buildings in relation to their durability, adaptability and energy conservation exists in the management of property estates. The assessment can be used in the decision making process on maintenance, upgrading, functional improvement, changes of use, energy conservation and potential savings.

## 6 CONCLUSIONS

Close collaboration with practitioners throughout this research project provided a valuable insight into the needs of practice with regard to the design and management of more sustainable buildings. This has informed the development of the DAEC Tool. Application of the Tool on selected higher education buildings and on a new building design demonstrated that the DAEC Tool can be applied to different types of buildings because it is designed as a framework which allows for modifications of the assessment criteria and related performance targets.

It is envisaged that the DAEC Tool will be used by a team of building professionals (e.g. architect, structural engineer, services engineer, quantity surveyor and contractor) and a client during the design of new buildings or in the management of existing buildings. To the building design team and client, the DAEC Tool serves as an aid in developing a design brief, in the building design process, and in the selection of design options. The design team and client need to agree about the proposed criteria and performance targets, which will then inform the design process and provide guidance in the selection of design options. When the DAEC Tool is applied in such a way, i.e. after the agreement between the designers and clients on the assessment criteria and performance targets, the subjectivity in the assessment process will be minimised. The same applies to the use of the DAEC Tool in the assessment of existing buildings. The DAEC Tool provides a generic assessment approach which can be adapted to different building types and performance targets.

The collaboration between researchers and practitioners in this research project contributed to the development, application and refinement of the research outcomes. Future research could focus on developing DAEC Tool versions for different types of buildings, and for client evaluation of building designs in the bidding process.

## 7 ACKNOWLEDGMENTS

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# MONITORING MOISTURE IN CONCRETE WITH AN EMBEDDED TRANSMISSION LINE

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**Summary: In monitoring concrete structures, long-term moisture monitoring is important as the moisture plays a pivotal role in the chemical reactions of the corrosion process. The paper describes the development of two versions of a low cost robust transmission line sensor operating at 1 GHz and capable of sensitive tracking. The sensor can be placed either directly into freshly cast concrete or can be embedded within a porous medium for pre-calibration prior to installation within existing structures. While the performance of the sensors is very encouraging, results show that knowledge of the relationship in moisture transfer between the porous media is essential for reliable moisture content measurement.**

**Keywords: moisture, concrete, transmission line**

## 1 INTRODUCTION AND BACKGROUND

Moisture related problems are a leading cause of degradation in many building materials. Despite the importance of measurement and monitoring, the accuracy, reproducibility and reliability of measurement is often limited especially where hand-held moisture meters are employed. Problems in measurement using hand-held instrumentation were extensively discussed previously, for example in Ahmet *et al* (1996). Remote moisture sensors are increasingly being used in conjunction with associated electronics and circuitry to monitor moisture. Various types of wood-sensor are commonly employed where the principle is based on the measurement of electrical resistance, which is a function of the moisture content. Tracking using a datalogger provides useful information about the variation in moisture content and examples of such sensors were extensively discussed in Dai and Ahmet (1999). Unfortunately, experience has shown that such sensors tend to be reliable for short- and medium-term monitoring. For long-term monitoring (many years) necessary for concrete structures, the authors vigorously sought and investigated alternative principles for measurement. A very promising solution is discussed in this paper.

Long term moisture monitoring can provide early warning of changes inside concrete structures that may lead to possible damage requiring remedial action. Extensive literature reviews concluded that there is an acute requirement in the construction industry for a reliable, non-destructive and cost-effective method to track moisture variations in concrete and to monitor the moisture content over many years or even decades (Parrot L. J., 1990) (Ahmet K., *et al*, 1999). It is recognised that while a range of methods exists for the monitoring of moisture in construction materials, whether based on the determination of moisture content or the relative humidity, these methods are predominantly aimed at either one-off measurements, or relatively short term monitoring over a few weeks or months. Measurements should be made at various key points within the concrete and not confined to surface or near-surface monitoring. Further, sensors should be capable of functioning reliably over the long-term without the need for re-calibration and must be low-cost to be employed in large numbers (Ahmet K. and Malan F. S., 2000).

The transmission line type sensor developed appears to fulfil the stringent requirements for long term monitoring. The remainder of this paper justifies the various configurations of sensors investigated.

## 2 METHODOLOGY

The design was based on the investigations and results obtained by calibration with time domain reflectometry (TDR)[1] measurements using a HP 8752C Network Analyzer. Permittivity measurements were done with a 5-conductor semi-coaxial transmission line cast into the material under test (Fig. 1). There are two versions of the final probe transmission line, one with and one without moisture absorbent material. Calibration was carried out using TDR and transmission velocity measurements at discrete frequencies for the transmission line to determine the relative permittivity of the material plotted against moisture content determined by oven drying and weighing.

[1] TDR is the measurement of the time elapse between transmission of an electromagnetic pulse along a transmission line and the reception of the reflection from the end of the line.

Volumetric moisture content measurements were used throughout and gravimetric results were converted to volumetric moisture content for calibration purposes[2]. This method is often used in microwave aquametry (Topp G.C. and Ferre P.A., 2001).

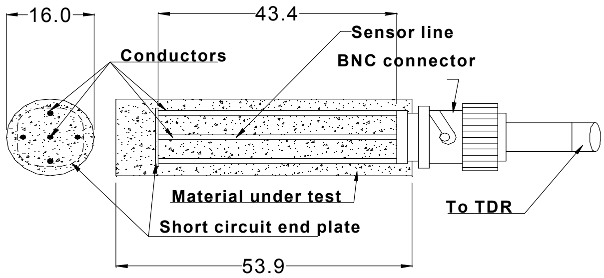


**Figure 1Calibration line cast into material under test (dimensions in mm)**

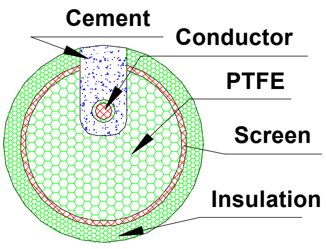


**Figure 2Line with cement paste filler around conductor**

The final probe design consists of a radio frequency oscillator (operating at about 1GHz) that uses the transmission line as the tuning device to determine the frequency. In this setup the frequency of oscillation is converted to an output voltage, which can be monitored by means of a voltage datalogger. The output voltage is calibrated in terms of moisture content. Energy consumption is minimal as it is supplied from an external source only when a reading is taken. Details were provided in Ahmet and Malan (2000).

One major drawback of the short-circuited closed coaxial line used in the initial calibrations is that the material under test has to be enclosed inside the "cage". Three ways were investigated to overcome this problem.

a) The material inside the cage is allowed to take up the moisture from or to release it to the surrounding structure to measure the moisture content (Fig. 1). This was the calibration setup.

b) The design is such that only part of the material in the cage is sensitive to moisture from the concrete (Fig. 2 shows that the rest of the volume is filled with PTFE).

c) The transmission line is designed in such a way that it will measure the moisture in the wall of the drilled hole i.e. it is sensitive to the material immediately surrounding the sensor (Fig. 3). In this case the electric field changes are affected by the material surrounding the sensor. The dielectric material (PTFE) inside the sensor remains unchanged.

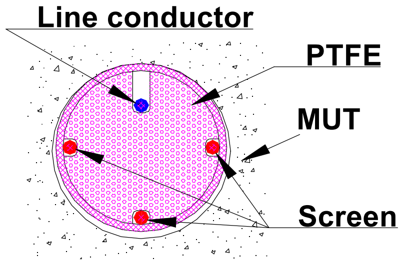


**Figure 3Off-set or "skew" line**

[2] The ratio of weight loss and therefore the volume of water lost to the volume of the sample.

# 3 MOISTURE IN CONCRETE

Although there are many types of concrete available, this discussion will be restricted to a common type, produced using "Ordinary Portland Cement" (OPC) mixed with aggregates of varying particle sizes and reinforced with steel.

The hardened cement paste that is obtained by mixing cement powder with water is characterised by a porous microstructure with high surface area (200 $m^2/g$ when fully hydrated, based on measurement by water adsorption) (Consolati G, *et al*, 2001). Because mature hardened concrete is porous it absorbs water quite readily, but with such a mixture of materials the pores can vary greatly in size from clearly visible air and/or water spheres, cracks and fissures to nanometre sized gel pores in the cement paste. The types of pores can be divided into two main categories namely:

a) Spherical pores and other larger pores, cracks and voids in the concrete mix.

b) Capillary pores and gel pores.

Aggregates used in the concrete mix can vary in porosity from highly porous material to virtually non-porous sand grains and crushed rock.

## 3.1 Spherical pores and cracks

Moisture can be trapped in residue material around the aggregate pieces and in cracks and fissures outside as well as inside the cement paste. The authors used electron microscopy to qualitatively investigate these effects. An example is shown in Figure 4. Trapped air in the mix forms spherical pores. The cement hydration process absorbs most of the water in the mix, but excess water will diffuse through the matrix and evaporate, leaving capillary pores in the matrix. The capillary pressure ($P$, measured in Pa) in a capillary tube (adapted from Lorenzo A.,1928) is given by:

$$P = 2\gamma / r \qquad \textbf{Equation 1}$$

where: $\gamma$ is the water/cement adhesion force (N $m^{-1}$)

$r$ is the capillary radius or hydraulic radius (m)

Equation 1 shows that the capillary pressure increases as the inverse of the radius. Clearly, for nanometre scale pores this represents a tremendous negative pressure (suction) that a capillary exerts on free water although the flow rate is extremely slow. The implication is that with an increase in moisture the larger pores are the last to fill up with water, conversely they are the first ones to become dry when there is a reduction in moisture content.

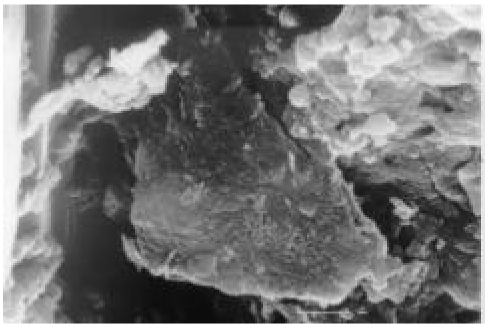

**Figure 4. SEM photograph of a polished cement paste surface. Scale 8000:1.**

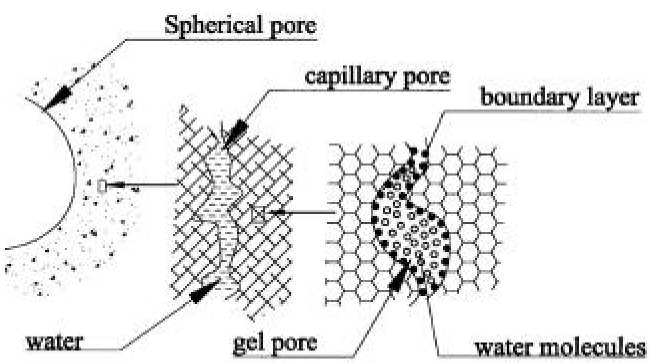


**Figure 5. Diagrammatic representation of the pore system in the cement paste**

### 3.2 Gel and capillary pores in the cement paste.

Figure 4 gives an impression of the range of pore sizes found in cement paste (possibly near a crack in this case). Figure 5 is a simplified diagrammatic representation of the pore system in the cement paste.

Gel pores are nanometre size pores (<10 nm), which form continuous pathways in the hydration product. The capillary pores (10 nm < $r$ < 1000 nm) are between the hydration product regions where excess water is trapped forming together with the gel pores a continuous network across the material (Consolati G. *et al*, 2001)(Garboszi E. J., Bentz D. P., 1992). The implications for the sensor design are discussed later.

## 4 MEASUREMENT OF MOISTURE IN CONCRETE

In the methodology used moisture content (MC) is calculated by determining how the relative permittivity $e_r$ of the material under investigation varies with MC. The water has a much higher $e_r$ than the dry material. Extensive TDR measurements were used for the calibration of the sensors to determine the reflection time from the end of the line in picoseconds and $e_r$ is calculated from:

$$e_r = (\text{measured TDR time/calculated reflection time for the line in free space})^2 \qquad \text{Equation 2}$$

### 4.1 Volumetric moisture content.

In this paper volumetric moisture content is used throughout, providing an ‘absolute moisture level’. Briefly the reasoning is that as concrete consists of a mixture of different materials, any meaningful measure of moisture in a mix of materials should be independent of the density of the material. That is why the measured moisture content (MC), defined in Eq. 3 below, is in terms of a percentage of total volume (absolute moisture level).

$$\text{MC} = 100\times(\text{total volume of water/ total bulk volume})\% \qquad \text{Equation 3}$$

In the rest of the paper absolute moisture content will be referred to as MC[3].

### 4.2 Relative humidity ($H_R$) as a measure of moisture content

Under normal environmental conditions, concrete structures are frequently subjected to running or standing water. Except for a small amount of trapped air even the mature concrete could be fully saturated with water. It is common practice to use measurement of $H_R$ inside a sealed cavity as an indication of moisture content or dryness of concrete. It was shown in section 3.1 that as the concrete dries out, the moisture becomes ‘concentrated’ into the smaller and smaller capillaries. The relationship between $H_R$ and pore size filled with water is given by the following formula of equation 4 (adapted from Bentz D.P., *et al* , 1999)

$$\ln(H_R/100) = -2V_m \gamma / rRT \qquad \text{Equation 4}$$

where

$H_R$ = relative humidity (%)

$V_m$ = molar volume of water ($m^3\,mol^{-1}$)

$g$ = adhesion force between water and solid ($N\,m^{-1}$)

$r$ = capillary radius (m)

$R$ = Universal gas constant ($J\,K^{-1}$)

$T$ = Absolute temperature (K)

At constant temperature in a homogenous porous solid, Equation 4 can be reduced to:

$$H_R = 100\, e^{-C/r} \qquad \text{Equation 5}$$

where $C = 2V_m \gamma / RT$ is constant for given values of $g$ and $T$.

Using Equation 5 it can be shown that $H_R$ will reach close to 100% when the largest capillaries containing water are still in the nanometre region. Equation 5 further shows that $H_R$ is a measure of the size of gel pores ($r$) still filled with water. $H_R$ measurement can be translated to moisture content when the total volume of pores smaller than diameter $r$ is known. Although $H_R$ is therefore a good qualitative indication of the “dryness” of the concrete, accurate measurement of relative permittivity $e_r$ is superior; it provides a more direct indication of moisture content and can measure effectively any level of moisture content.

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[3] Volumetric moisture level can be converted to gravimetric moisture level by dividing the volumetric moisture level by the density of the dry material in $g.cm^{-3}$.

### 4.3 Change in relative permittivity of concrete with moisture.

The dipolar character of water molecules results in a relative permittivity much higher than that of the dry material (about 76 compared to 4 for the dry material). As already mentioned moisture content can therefore be very effectively determined by measuring the relative permittivity of the material under test. Figure 6 are typical results showing the variation of relative permittivity as a function of the moisture content for 50% sand/cement mortar and cement paste. Measurements were carried out with time domain reflectometry (TDR) using the fully embedded transmission line shown in Fig. 1. The w/c ratio was 0.5 for both the cement paste and the mortar. Although the best fit is a fourth order polynomial, a straight-line fit as shown is good enough for practical purposes.

### 4.4 Advantages and disadvantages of the “skew line” sensor.

Figure 3 shows the position of the line conductor in this type of sensor, which determines how much of the electromagnetic (EM) field lines penetrate the material under test (MUT). When the line conductor is placed nearer to the outside edge, the EM field lines will penetrate deeper into the MUT. With the EM field penetrating more of the surrounding material the sensor will be more sensitive to changes in moisture content. However, it would also be more sensitive to the presence of disturbances like steel reinforcement and other discontinuities in the concrete. If the sensitivity is reduced (the conductor moved towards the centre of the probe) the probe will not detect very low moisture levels. Optimum configurations are currently being investigated.

For unambiguous calibration the distance between the line and the MUT must be constant. The probe is positioned with the line conductor side of the sensor pushed firmly against the side of the hole to achieve this.

### 4.5 Advantages and disadvantages of transmission line with absorbent element.

The probe in Fig. 2 can be calibrated accurately for moisture content in the absorbent material of the probe. The probe is also unaffected by nearby metallic objects (steel reinforcement) or proximity to a sudden break or change in the MUT. It can be tied to the steel reinforcement if the probe is to be cast permanently into the concrete or placed close to the surface or edge of the MUT.

The main disadvantage of using a porous material in the sensor that absorbs moisture from the concrete under test, is the long time delay to reach equilibrium. This arrangement is therefore suitable for long term moisture monitoring and not for a quick assessments of moisture levels.

Equation 1 shows that the moisture content in the MUT and the probe absorbent depend on the relative capillary sizes. The moisture content is also be a function of the ratio of porous and non-porous materials in the MUT. The porosity of aggregates varies and could be of the same order of magnitude as the porosity of the cement paste. At very high levels of moisture close to saturation the probe material is likely to be saturated before the MUT if the latter contains cracks and voids.

Using the probe on a different material to that in the line requires knowledge of the relative pore sizes of the MUT. From equation 1 it is evident that cement paste, which has pore sizes in the nanometre region, could absorb all the moisture in the MUT with larger pores, indicating a much higher moisture content than is actually the case. This type of sensor is material specific, which means it can only be used to measure moisture in concrete if the filler material is cement paste.

## 5 TEST RESULTS

TDR measurements were carried out on the calibration line (Fig. 1) using the HP 8752C Network Analyzer. Results showing the correlation between the relative permittivity and moisture content are shown in Figures 6. The shorter dotted line is for mortar with 1:1 ratio of sand and cement. In both materials the w/c ratio was 0.5. Notice the excellent correlation between the relative permittivity and the moisture content. Further, the calibration line for the mortar is very close to that of the cement paste.

Although the best fit trend line is a fourth order polynomial, the straight line approximation is good enough in both cases for most practical purposes, where $R^2 > 0.995$. The small pore sizes imply that the diffusion rate through the material under test would be very slow. With a drying out cycle the moisture at the core could have been higher compared to a position near the outer surface, in which case the relative permittivity would be measured higher than it should be. A wetting cycle should show the opposite effect. The actual measurement of relative permittivity ($e_r$) could be expected to be half way between the cycles. Even if a relatively long time is allowed to reach a homogeneous moisture distribution, the fact that diffusion can only take place when there is a moisture gradient in the material means that some hysteresis effect will always be present. Another reason why the wetting and drying cycle may differ is the fact that there are clumps of hydration product (gel) in the material surrounded by larger gaps (see Fig. 4). The result is that on a drying cycle some moisture may be trapped in the nanometre pore system of the gel. The trapped moisture will eventually dry out as the humidity decreases, but the process is very slow. The $e_r$ for the dry material in both cases is very close namely 4.96 and 4.98 for the cement paste and the mortar respectively. Using the linear approximation for $e_r$ at 10% moisture level the values are 7.78 and 7.70 respectively, i.e. excellent agreement for the two materials.

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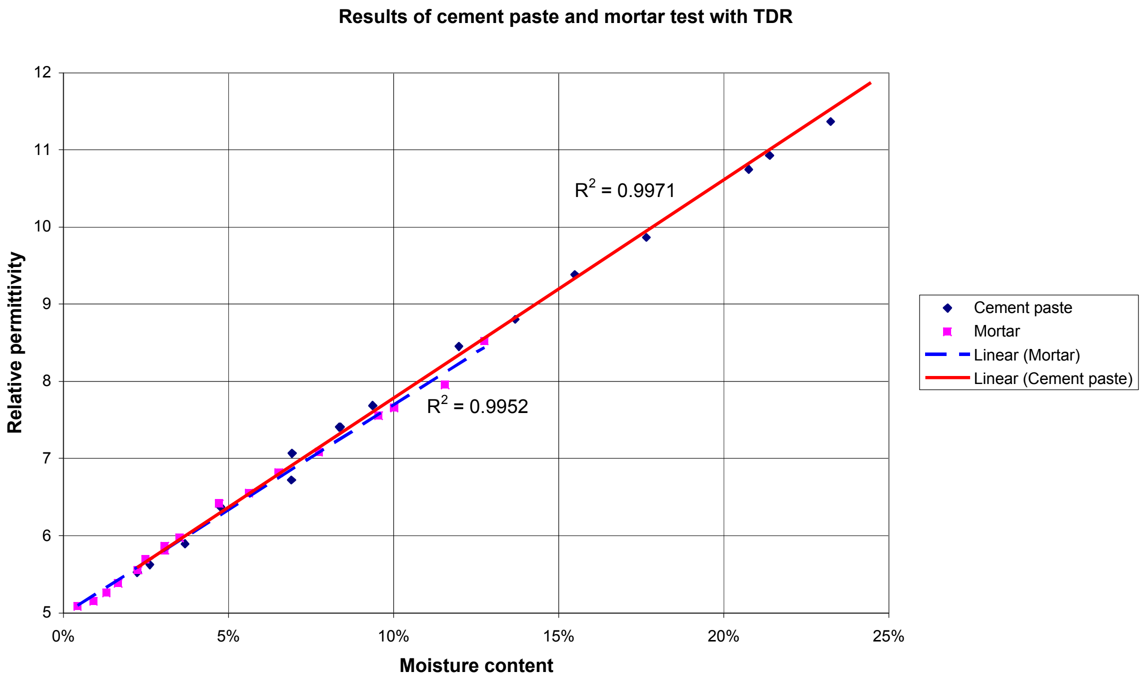


**Figure 6 Test results with cement paste and mortar dielectric with TDR**

The results also reveal that the saturation level for the cement paste is about twice as high as for the mortar, showing that the sand in the mortar with a 1:1 sand to cement mixture ratio took up a negligible amount of moisture (tested at normal atmospheric pressure). This is in accordance with expectations.

Finally it is important to realise that TDR measurements are broad band frequency measurements. The relative permittivity changes with frequency so that the value obtained in Figure 6 with TDR is an average of all the relative permittivity values in the frequency band 300 kHz to 6 GHz employed on the instrument.

## 6 CONCLUSION

This paper has described a method to determine the moisture content of concrete as part of a continuous monitoring programme. Used in conjunction with sensors to detect for example physical movements, the very useful information will give early indication of deterioration of the structure.

There are a number of configurations in which the probe could be employed and two possibilities have been summarised in this paper. Present work is being carried out to find the optimum solution for each application. Indications are that there is no single optimum solution that will satisfy all applications or moisture conditions simultaneously. As an array of sensors would normally be used in a large structure, these sensors could be a mixture of types with different dynamic ranges to cover MC from dry to saturation levels, depending on the range of interest.

This transmission line MC probe appears to be a promising technique to determine the absolute moisture content at a specific location in the concrete. The “skew line” design has the potential to measure MC up to virtually full saturation levels. In this arrangement no time delay would have to be allowed for the probe to absorb moisture. With the relative low cost of the probe and the fact that it can be left in situ for many years due to its very robust nature, it appears to overcome many of the problems with present day techniques. Despite this, knowledge of the limitations and nature of moisture content equilibrium in various concrete types or other porous media is required to determine the absolute moisture content. Although the intention was to design sensors to be mounted in hardened concrete, both types of sensors described could be embedded in the concrete when it is cast in order to monitor the setting and drying of the concrete and then left in situ for long term monitoring of the structure.

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# A Markov Approach In Estimating The Service Life Of Bridge Elements In Sweden

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**Summary: The service life of Swedish road bridges has previously been studied by collecting inspection reports and other significant information from 353 bridges. A total of 3747 bridge inspection remarks were gathered and the type and cause of damage were stated and each element was given a condition class. This information was then inputted into a relational database. Even if the results are not significant for the entire country, they give a clear indication of the general condition of the bridges in certain parts of Sweden.**

**It has been shown that four structural members were responsible for almost half of the deterioration reported. These structural members are the edge beams, supports, slope and embankment ends and deck slabs. One structural member, the parapet, was mainly damaged by vehicle accidents.**

**Using various search criteria, together with the relational database, a detailed analysis of the real service life of bridge elements can be performed. However, by increasing the number of specific search criteria in an analysis, the resulting population decreases. Examples of search criterion are bridge generation, element type, damage type and cause. Deterioration of bridge elements can be analysed numerically using the Markov chain theory. The deterioration of a particular structural member must be defined by a number of states, in this case given by the assessed condition classes. A state vector that gives rise to a new state after multiplication by a transition probability matrix defines the states of a population of elements.**

**It is demonstrated how a transition probability matrix can be numerically determined to describe the deterioration process of a bridge element from data in the relational database. The numerical method used, is based on an iterative stepwise combination of the matrix elements until the error between a known deterioration average curve and a curve given by the Markov chain is minimized. The method is computationally demanding for small steps but will, with larger steps, quickly converge towards an approximation close to the transition probability matrix. It is also demonstrated how the remaining service life of a bridge element can be estimated by studying the variation of the state vector with bridge age.**

**Keywords: Bridges, Markov chain, Degradation, Estimated Service, Condition Class.**

## 1 INTRODUCTION

In Sweden, the importance and need for research in the field of maintenance, operation and repair of bridges was realised already in the mid-sixties. By the beginning of the 1980's, the *Swedish Transportation Research Delegation* appointed a special committee with the task of assessing Research and Development (R and D) needs and co-ordination of the R and D projects on maintenance and repair (VTI 1985, 1989; Ingvarsson & Westerberg 1986). The research programme "The Durability, Management Repair and Life Cycle Costs of Concrete Structures", was conducted at the *Department of Structural Engineering* at *the Royal Institute of Technology (KTH*) in Stockholm. The programme was initiated by the *Swedish National Road Administration* in 1991. The objective of the programme, which consisted of seven different projects, was to study deterioration of concrete structures, defect detection methods and effective and economical systems for design, redesign and assessment of structures for the traffic infrastructure. The results of the above-mentioned R and D have contributed to a well-needed manual in the field of operation, maintenance and repair of structures (Silwerbrand & Sundquist 1998).

The present paper discusses one of the possibilities offered by the results presented by (Racutanu 2000). The aim of this research was to develop different deterioration models at the bridge component level for certain damage types in certain service condition. All bridge technical and historical information available from 353 bridges in the different databases of the

*Swedish National Road Administration* was inputted into a relational database. Service life models in combination with a relational database containing actual durability information can provide a tool for assessing and predicting damages. If such a model could be achieved, it would give bridge owners a precious tool in the planning of maintenance works, feedback in the future detailing of certain bridge components and for future codes and regulations.

Bridges are an important part of a nations road system. Therefore, maintaining and repairing existing bridges are a major economic concern for many governments and local authorities around the world. Many bridges are built with technical codes and demands that are outdated today due to the longer service life demands. It is generally expected that during their service life, bridges can fulfil certain demands such as traffic safety, continuous traffic flow and a designed load bearing capacity. Regular and systematic inspection of the existing bridge stock should be performed in order to verify that such demands are met at all times.

With the 1944 nationalisation of the public roads in Sweden, information about most of the regular bridge inspections have been carefully documented and filed by the *Swedish National Road Administration* in different archives.

The functional condition of a damaged structural member (an element) is described with the help of condition classes at the time of inspection. The development of the assessed condition class in time can be a way of predicting the service life of bridge components, contributing to cost-effective bridge management and improved bridge component design in the future.

Damages are rated in condition grades on a scale between 0 and 3. Sweden has many different local climate conditions. In the north of Sweden there are few yearly frost-cycles, mainly because the temperature is always below zero degrees Celsius throughout the winter season while it can oscillate around zero degrees in central and southern Sweden, leading to several frost-cycles. This implies a need for a better frost resistance performance of the concrete in the central and southern parts of the country. De-icing salts are used on many roads during the cold season of the year. Because of the various climate conditions in Sweden, bridges in the central and southern part of Sweden are especially exposed to large amounts of de-icing salts

## 2 CASE-STUDY RESULTS

There were a total of 3747 bridge inspection records from the 353 investigated bridges. There were 2980 records with inspection notations and 767 inspection records where the structural members were considered flawless at the time of inspection.

Although 353 bridges represent about 2 % of the bridge infrastructure, they give a clear indication of the general condition of the bridges in certain parts of Sweden.

### 2.1 Damage causes and types on the structural member level

Fig. 1 presents the inspection for the different structural members of the bridges. The percentage division of all the damage remarks in the case-study, even those damages that have been given the condition class 0, i.e. no deterioration, show a remarkable similarity, independent of where the bridge is located in Sweden. The most interesting result in the context was the fact that the structural members *(Edge beam*, *Support* and *Deck slab)* together were responsible for 44 % of all distresses, regardless of the year of construction, type and cause of damage. The structural members *(Slope and embankment end* and *Parapet)* together were responsible for 23 % of all remarks. The restoration of embankment ends is a frequent maintenance problem for the Swedish bridge management. Anecdotally, steep slopes and embankment ends, especially for older bridge stock, have such high inclinations that it seems like the bridge designers wished to defy the law of gravity.

On most Swedish bridges the parapet is fixed to the edge beam. This is, in many cases, an unfortunate solution. The structural member *(Parapet- 13 %)* is more frequently involved in collisions, and this can lead to a domino effect where the edge beam concrete (*Edge beam- 21 %)* is damaged. Worth noting is that the structural member *(Primary load-bearing element- 6 %)* has managed fairly well in the investigation. In fact, less than 1 % of all reported damages were assessed condition class 3 signifying that, at the time of inspection, the structural member has entirely lost its designed function.

The classification of damage causes follows the system used by the *Swedish National Road Administration*. It is relatively simple yet effective way to define the entire degradation process

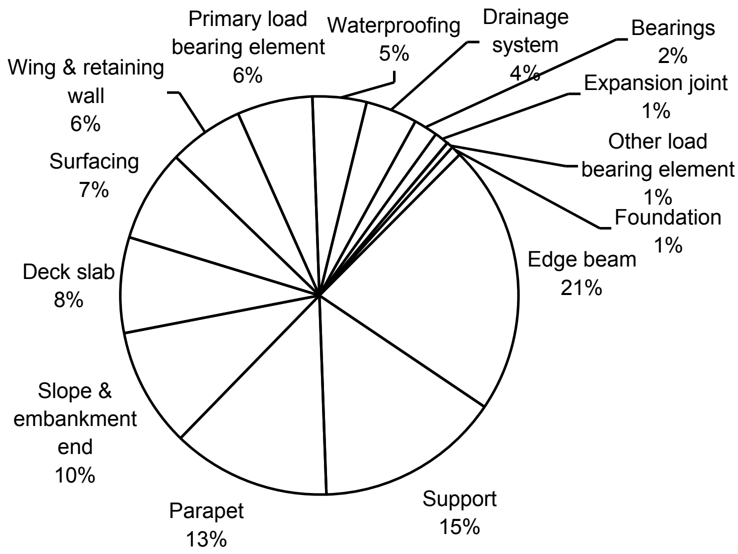


**Figure 1. A total of 3747 damage remarks, all condition classes, percentage division by the bridges' structural members.**

without being too complicated for the user in the field. First, one of the six possible primary causes for the damage must be reported. These are: *Defective construction*, *Service condition*, *Environmental action*, *Accident*, *Defective maintenance*, *Defective design* as seen in Fig. 2. As the primary cause is established or assessed, the possible secondary and tertiary causes have to be reported. Surprisingly, almost three-quarters (76 %) of all 2850 damages reported in the investigation were assessed to *Service condition* and *Environmental action*. It is obvious that these two damage causes should be examined closer as to their secondary and tertiary causes. Even more surprising was the fact that less than one percent was accredited to *Defective design* and only 5 % accredited to *Defective construction* as the main causes of the assessed damage.

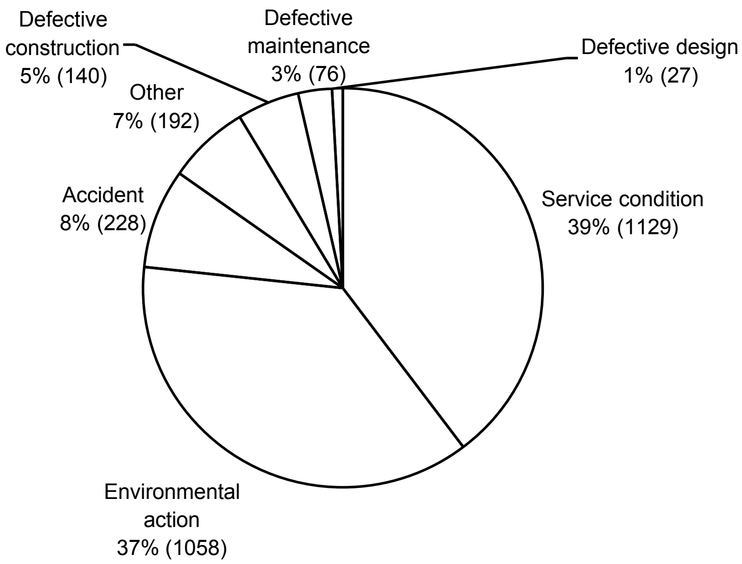


**Figure 2. The 2850 damages in the investigation divided by their primary cause.**

Vehicular accidents happen on bridges and will always happen. The bridge designers must minimise the risk of structural damage to the bridge. Even if accidents represent only eight percent of the damages shown in Fig. 2, it does not exempt the damage cause from suggesting design improvements. For example, if the 228 reported accidents, all were caused by the secondary and tertiary causes *Collision*. The parapet placement could be revised in such a way that the deflector rail is separated from the parapet or that the parapet should not be cast into the edge beam concrete as the standardised design solution of the parapet in Sweden recommends today. The primary load bearing elements, if hit and seriously damaged in a collision, can jeopardise the entire structure. A total of 1129 damage remarks were directly assessed to *Service condition* whereof 62 % of the damage remarks are assessed as loading damages and out of these, only 2 % were accredited to *Overload*, which seems reasonable. Traffic loads have steadily increased over the years and the elderly bridge-stock was never designed to carry today's heavy traffic. Erosion is the cause of 13 % of the damage remarks. Environmental action has been used extensively as the primary cause of damage. There is however a risk that this damage cause has been abused by the inspectors.

"Blame it on the nature" could hide many of the real causes such as hidden faults, poor detailing or poor workmanship. An interesting fact in the entire investigation was the low percentage of damage caused by Alkali-Silica Reaction (ASR).

## 2.2 Bridge Element Analysis (BEA) Relational Database

With the help of the *Bridge Element Analysis, BEA,* relational database, it is possible to investigate the damage picture by changing some search criteria. In this section, investigation of the over-representation of specific structural members takes place. The search possibilities of a well-designed relational database are enormous. Six structural members were studied. These were the *Edge beam*, *Support*, *Deck slab, Wing and retaining wall, Slope and embankment end* and finally, the *Parapet*. In Table 1, the results are presented, using the structural construction type of bridge as search criteria.

**Table 1. Distribution of all remarks on six of fourteen structural members.**

| *Structural construction type* | *Edge beam* | *Support* | *Deck slab* | *Wing & retaining wall* | *Slope & embank-ment end* | *Parapet* | ***2.3 2.4*** |
|---|---|---|---|---|---|---|---|
| Slab bridges | 21 % | 13 % | 9 % | 1 % | 10 % | 11 % | 65 % |
| Beam bridges | 21 % | 16 % | 6 % | 2 % | 5 % | 9 % | 59 % |
| Slab frame bridges | 22 % | 17 % | 7 % | 9 % | 10 % | 14 % | 79 % |
| Beam and slab bridges | 23 % | 6 % | 6 % | 9 % | 19 % | 14 % | 77 % |

# 3 PREDICTION OF DETERIORATION

It can be very difficult to find two similar bridges with identical conditions of service life. It is possible to minimise the differences, for example through following structures on a geographically close road section. If a geographically close road section is chosen, the Average Annual Daily Traffic (AADT), the climate, the road environment and the standard of the road are relatively similar. Also the same personnel perform the inspections during longer period of time, and many of the successive bridges are contemporary, build during the same period of time, i.e. the same regulations, codes and construction techniques have been applied**.**

With gathered inspection information and in constant contact and communication with a number of experienced bridge inspectors, it soon became obvious that bridges do not just break down, accidents

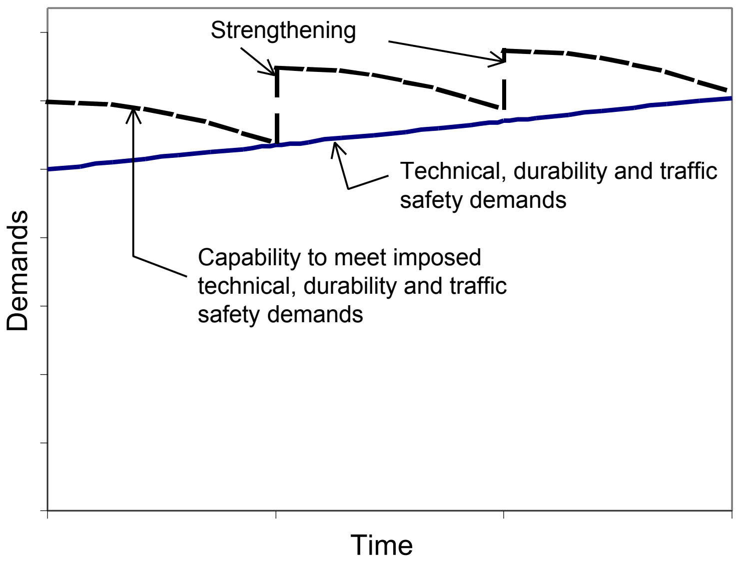


**Figure 3. Increased demands in time of the load carrying capacity, durability and traffic safety of structures.**

exempted. This is also true about buildings and facilities (Watson 1996). It is their structural members and components that deteriorate and single, or in interaction, finally can bring a whole structure down. The increase of the maximum allowable load of the heavy vehicles and the demand for a more cost-effective transport system has also affected design codes in time. These increasing demands have led to an increased load capacity, traffic safety features, such as free height, or improved function of bridges in time. For this reason, engineers have often built in safety factors, or extra strength, in the structures in the design stage. These safety factors have varied in time, depending upon the building and design codes in use at the time. Future repair or replacement of a structural member will be conducted after the design codes in use at the certain time, thus increasing the load bearing capacity and the resistance to deterioration as considered at that particular time. The principle is presented schematically in Fig. 3.

### 3.1 Different approaches in predicting damage growth

A number of different prediction methods using inspection information on the condition and time in service of materials and machine components have already been developed for other industries and management systems. For example, deterioration models for road pavements incorporated in life cycle models have already been introduced in some maintenance and management systems. Such examples are the *HDM* – series life cycle models for roads (Robinson et al 1998). Two main classes of deterioration models can be used, probabilistic and deterministic. The probabilistic models predict damage growth as a probability function of a range of possible condition states. In a deterministic model, the condition is predicted as a value based on one or more mathematical functions. These functions are based on observed or measured deterioration. (Haas *et al.* 1994) has categorised condition projection into four main categories:

- Subjective
- Purely mechanistic
- Regression
- Mechanistic-empirical

The subjective condition projection can be used when pure experience is used in creating prediction models. Purely mechanistic prediction models are based on some primary response or measurable parameter for the examined subject, e.g. crack width in concrete. Regression prediction models can be put to use when the dependant variable of observed or measured damage growth is related to one or more independent variables, such as year of construction, environmental factors, structural members and their interaction. The mechanistic-empirical prediction models use defined equations based on mechanistic principles, in order to relate a dependent variable to measured damage growth. Thereafter, regression analysis is used to determine the coefficients and parameters of the equation. In this paper, the information gathered during the case study will be used in a probabilistic method using a Markov chain method.

## 4 THE MARKOV CHAIN METHOD

The Markov chain is a convenient tool for estimating the service life of bridge components (Jiang & Sinha 1989). The application of the Markov chain technique in estimating the service life of components in technical systems has been used in a number of different areas, such as the deterioration of sewer systems (Abraham & Wirahadikusumah 1999). The results in the form of numerically determined deterioration curves proved to give good approximations.when compared to deterioration curves based on experience and expert opinions. A preliminary investigation of possible numerical implementations of the Markov chain method for estimating the service life of bridge components has been carried out by (Ansell 2001).

### 4.1 Matrix formulation of the Markov chain

A deterioration function based on a Markov chain is used here to approximate an average condition rating at time *t*, estimated by a regression function *Y*(*t*). The accuracy of this approximation depends on the step length taken during numerical calculation of matrices within the Markov chain so that:

$$E(t,\mathbf{P}) \cong Y(t) \tag{1}$$

The values of condition ratings $E(t,\mathbf{P})$, estimated by a Markov chain, is given (Jiang and Sinha 1989) by the matrix and vector multiplications:

$$E(t,\mathbf{P}) = Q(t)\cdot R^{\mathrm{T}} = Q_t \cdot R^{\mathrm{T}} = Q_0 \cdot \mathbf{P}^t \cdot R^{\mathrm{T}} \tag{2}$$

Where superscript $^{\mathrm{T}}$ denotes transformation. The number of objects at each state at a certain time is expressed by a state vector $Q(t)$, thus providing a damage index distribution. The condition rating at age $t$ is calculated from the initial condition $Q_0$ at $t = 0$ by $t$ times multiplication by a transition probability matrix **P**, i.e. a chain multiplication. The deterioration is expressed in terms of discrete condition states. A four-graded scale is used, where index 0 to 4 defines the condition of the studied objects. Degradation index 0 represents the best condition, the initial condition state, while index 3 defines the limit state at which the service life is reached. Index 4 represents the post limit state. In the above matrix formulation, the vector $R$ is a vector of condition ratings which connects the states to the condition rating scale, here:

$$R = \begin{pmatrix} 0 & 1 & 2 & 3 \end{pmatrix} \tag{3}$$

An initial state vector is thus:

$$Q_0 = \begin{pmatrix} 1 & 0 & 0 & 0 \end{pmatrix} \tag{4}$$

The relationship between state vectors as a function of age is:

$$Q_{t_0+t} = Q_{t_0} \cdot \mathbf{P}^t \tag{5}$$

An average transition probability matrix **P** is in the following denoted by $\mathbf{P}_{n\text{-}m}$, and is valid from year $n$ to $m$. The number of years $N$ over which the average of the transition elements within the matrix is taken is given by $m=n+N$. The chosen condition rating scale gives:

$$\mathbf{P} = \begin{bmatrix} p_1 & 1-p_1 & 0 & 0 \\ 0 & p_2 & 1-p_2 & 0 \\ 0 & 0 & p_3 & 1-p_3 \\ 0 & 0 & 0 & 1 \end{bmatrix} \tag{6}$$

The three diagonal transition elements $p_1$, $p_2$ and $p_3$ are the probabilities for the deterioration of a bridge component to remain in state 0, 1 or 2 when the component ages one time period, which in this case is one year. The diagonal element 1 in the fourth row and column restricts the values of condition ratings to 3, i.e. the limit state. The elements 1- $p_1$, 1- $p_2$ and 1- $p_3$ on the superdiagonal are the probabilities for the deterioration to advance one state as the bridge component ages one year. As a component will either remain at the same state or proceed to the next state in the next time period, the row sum of **P** must always be 1.

Thus, by using the Markov chain method, the following assumptions have to be made: The condition of the studied objects cannot be improved (repaired), and the condition state can either remain the same or shift to a higher within the next transition period. In reality, structural members are repaired and they can shift in one step to the third or higher condition state. The first assumption makes the model accurate until major repairs are made, usually after approximately 30 years on bridges in Sweden. It also implies that the transition probabilities below the diagonal probabilities are zero. The second assumption can be considered reasonable if all accident related damages are exempted. As an example, an edge beam or a parapet on a bridge can be destroyed on the first day of their service life by a vehicle collision. If the assumption is taken, all the probabilities above the superdiagonal in **P** are zero.

The matrix elements $p_1$, $p_2$ and $p_3$ are determined from the known relation $Y(t)$ by solving the non-linear minimisation problem:

$$\min \sum_{t=1}^{N} \left| Y(t) - E(t,\mathbf{P}) \right| \tag{7}$$

Where $0 \le p_i \le 1 \text{ for } \mathrm{i} = 1,2,3$ and $N$=3. This is done using a simple algorithm, which combines the matrix elements while keeping count of the error given by Eq. (7). The combination of $p_1$, $p_2$ and $p_3$ that provides the least error is the solution. The method is time consuming for small steps, and it is recommended that a more sophisticated numerical method be used in practice.

### 4.2 Example – deterioration of edge beams

To demonstrate the use of the Markov chain method, a population of edge beams from 21 bridges situated within the *Västmanland* in the south-western part of Sweden, is considered. The data consists of inspection data obtained from the bridge database maintained by the *Swedish National Road Administration*. Only inspection remarks prior to the first repair are considered, giving 21 series of data, each representing edge beams from one bridge. To each series, an exponential curve of the form:

$$Y(t) = a_0 \left( e^{a_1 t} - 1 \right) \tag{8}$$

was fitted in the least-square sense. These curves are shown in Fig. 4, where the median curve with respect to the constant $a_0$ is marked separately. The median curve, defined by

$$Y(t) = 0.100 \left( \mathrm{e}^{0.112t} - 1 \right) \tag{9}$$

**Table 2. Matrix elements of P for the approximation $E(t,\mathbf{P})$ of $Y(t)$ given in Eq. (9).**

| *Age of bridge elements (yrs.)* | $p_1$ | $p_2$ | $p_3$ |
|---|---|---|---|
| 1-3 | 1.00 | 0.00 | 0.00 |
| 4-6 | 0.95 | 1.00 | 0.00 |
| 7-9 | 1.00 | 0.95 | 0.35 |
| 10-12 | 1.00 | 0.80 | 0.15 |
| 13-15 | 0.95 | 0.90 | 1.00 |
| 16-18 | 0.95 | 0.85 | 0.85 |
| 19-21 | 0.90 | 0.85 | 0.90 |
| 22-24 | 0.90 | 0.80 | 0.75 |
| 25-27 | 0.80 | 0.65 | 0.70 |
| 28-30 | 0.45 | 0.35 | 0.55 |
| 31-33 | 0.00 | 0.00 | 0.00 |

is in the following used to represent the average deterioration of the edge beams.

A Markov chain approximation $E(t,\mathbf{P})$ to the average deterioration curve of Eq. (9) has been determined using the method described above, with steps of 0.05. In Table 2, the matrix elements of **P** are given as average values, valid for age intervals of three years. The resulting function is shown in Fig. 5 where Eq. (9) is plotted for comparison.

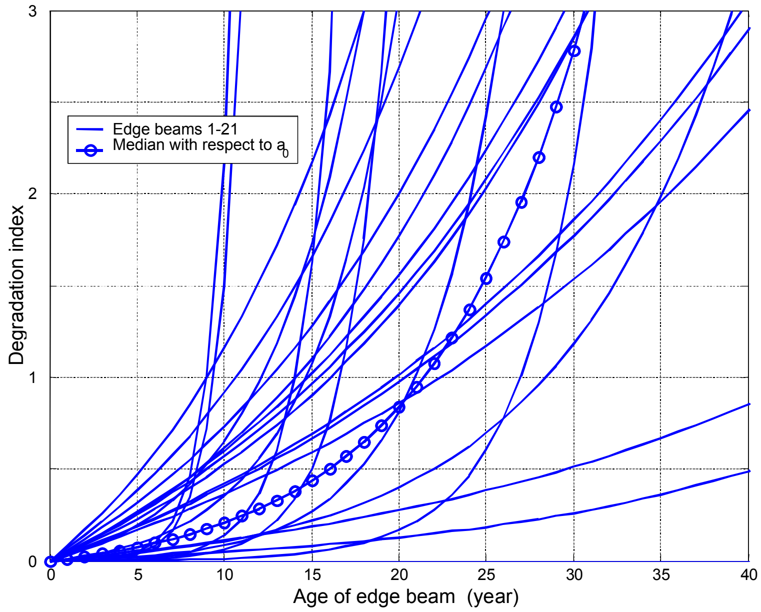


**Figure 4. The variation of the degradation index for a population of 21 edge beams.**

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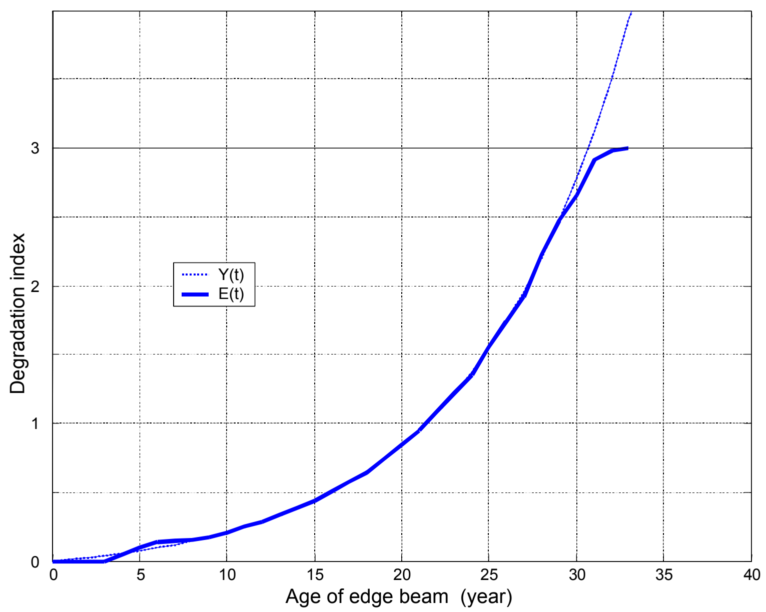


**Figure 5. The condition rating $Y(t)$ in Eq. (9) approximated by $E(t) = E(\mathbf{P},t)$ from numerical Markov chain calculations.**

As follows, the calculation of the estimated average state vectors is demonstrated in Eqs. (10-17). The matrix operations are for an average population of edge beam elements at 21, 22, 23 and 24 years of age. Two of the transition probability matrices are used in the calculations, since one is valid over the age range of 19-21 years and another over 22-24 years. The four calculation steps are also shown graphically in Fig. 6. The calculation steps that qives $Q_{21}$ and $E_{21}$ from $Q_{20}$ are:

$$Q_{21} = Q_{19} \cdot \mathbf{P}^2_{19-21} = Q_{20} \cdot \mathbf{P}_{19-21} = \begin{pmatrix}0.510 & 0.244 & 0.135 & 0.110\end{pmatrix} \quad (10)$$

$$E_{21} = Q_{21} \cdot R^T = 0.846 \approx 0 \quad (11)$$

Note that the sum of the elements in each state vector always is 1 and that the value of condition ratings, $E(t,\mathbf{P})$, is always rounded to the nearest integer towards minus infinity. The process is repeated for the following year, giving:

$$Q_{22} = Q_{20} \cdot \mathbf{P}^2_{19-21} = \begin{pmatrix}0.459 & 0.258 & 0.158 & 0.124\end{pmatrix} \quad (12)$$

$$E_{22} = Q_{22} \cdot R^T = 0.947 \approx 0 \quad (13)$$

For the third and fourth year, the chain of matrix multiplications is continued by multiplications with $\mathbf{P}_{22-24}$ , as this average transition probability matrix is valid from 22 years of age. This gives:

$$Q_{23} = Q_{20} \cdot \mathbf{P}^2_{19-21} \cdot \mathbf{P}_{22-24} = \begin{pmatrix}0.413 & 0.253 & 0.170 & 0.163\end{pmatrix} \quad (14)$$

$$E_{23} = Q_{23} \cdot R^T = 1.084 \approx 1 \quad (15)$$

and

$$Q_{24} = Q_{20} \cdot \mathbf{P}^2_{19-21} \cdot \mathbf{P}^2_{22-24} = \begin{pmatrix}0.372 & 0.243 & 0.178 & 0.206\end{pmatrix} \quad (16)$$

$$E_{24} = Q_{24} \cdot R^T = 1.218 \approx 1 \quad (17)$$

Note that the state defined by $E(t,\mathbf{P}) = 1$ is reached at an average age of 23 years since $E_{21} = 1$ .

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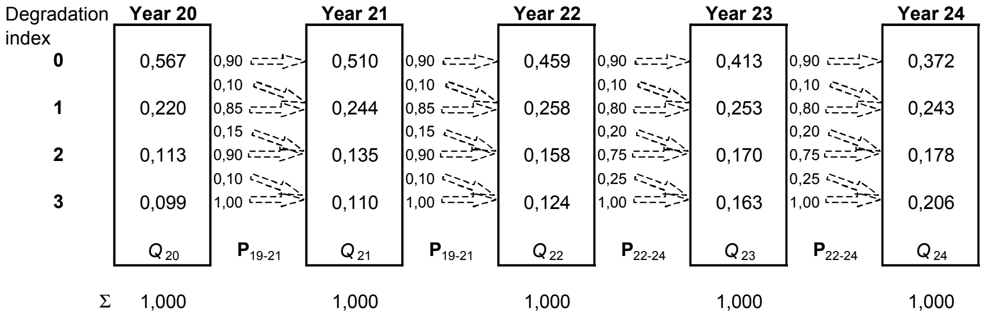


**Figure 6: Illustration of the matrix and vector calculations from Eqs. (10–17).**

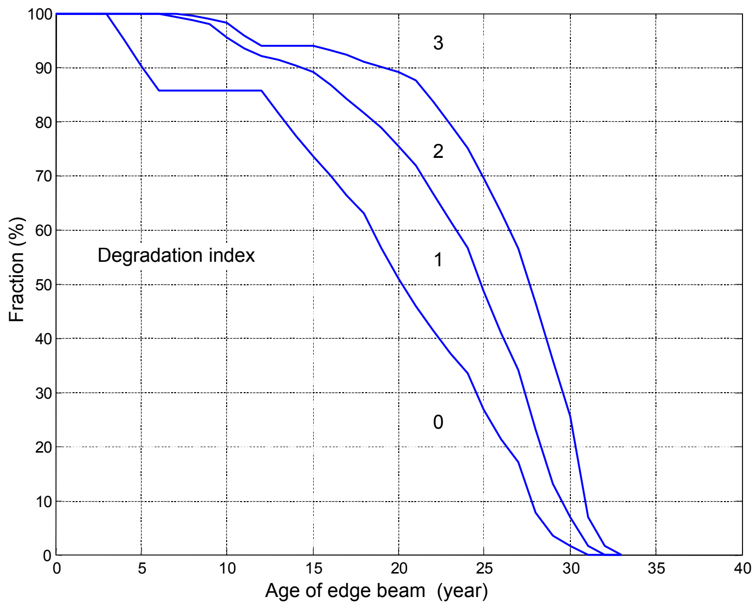


**Figure 7: The variation of the state vector, calculated using the diagonal transition elements given in Table 2.**

The state vectors contain information on the distribution of the edge beams of a random population, over the degradation indices 0-3. The variation of this distribution is shown in Fig. 7, which illustrates how additional information on the expected service life can be obtained from calculations using the the Markov chain method.

As an example, it can be seen that 50% of the edge beams are expected to have reached their maximum service life at an age of 27 years while the remaining 50% is expected to function for another 6 years. Thus, the described Markov approach makes it possible to extract information beyond what is possible from an analytical deterioration curve, obtained through ordinary curve fitting.

## 5 CONCLUSIONS

Important information on the bridges of Sweden has been collected in a relational database. The results give a clear indication of the general condition of the bridges in certain parts of Sweden. In order to predict the future deterioration of bridge components, numerical modeling combined with correctly chosen search criteria must be used. By increasing the number of specific search criteria in an analysis, the resulting population decreases. Therefore, the resulting population obtained must be representative of the bridge elements studied. The curve-fitting technique employed, is based on tracing all bridge components within a population until failure occurs. This technique will be further developed within this project.

The transformation matrices determined using the Markov chain technique can be used to obtain additional information on a population of deteriorated bridge components. One important example is the distribution of a population of bridge components over condition classes. Such information is not possible to obtain directly from an average deterioration curve fitted to a collection of data. The simple numerical algorithm that was used for solving the non-linear minimization is uncomplicated and safe but rather inefficient. In practical applications using the Markov based method, the computational effort will mainly consist of finding elements of transformation matrices. The computational efficiency can therefore be greatly improved if an advanced numerical minimization technique is used. This will be one of the main topics of the further research within the field.

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# Solutions And Strategies To Upgrade The Existing Housing Stock

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**Summary: The Netherlands is faced with a major undertaking in the period 2000 to 2010. In view of the backlog which has arisen in the quality of facilities and housing stock, the redevelopment of city-centre areas in particular, specifically the restructuring of postwar residential districts (1945 – 1960), requires a large-scale integrated approach.**

**This paper gives insight into the building volume within the housebuilding sector, the quality of the existing housing stock, the government's policy with respect to housing, sustainable building and urban restructuring.**

**An outline is given of the main strategies employed to restructure residential districts built in the period between 1945 and 1965.**

**The strategies have been divided into solutions for parcelization with low-rise buildings and for districts with stacked housing complexes. An example of a recently implemented practical experiment is described for each strategy.**

**Keywords: Sustainability, existing housing stock, recondition, reconstruct, partial demolition**

## 1 INTRODUCTION

The Netherlands is faced with a major undertaking in the period from 2000 to about 2010, the redevelopment of city-centre areas. In addition to the redevelopment of industrial estates, a sizeable part of this undertaking consists of restructuring residential districts built in the period 1945 to 1960.

This latter part, the restructuring of postwar residential districts is gone into in greater detail in this paper with respect to the problems, policy, solutions and strategies and experiences with practical experiments.

Before going on to discuss these issues, firstly a picture will be sketched of the situation concerning the quality of the existing housing stock in the Netherlands.

A picture which gives insight into:

1. the extent of the building volume in the housebuilding sector;
2. the quality of the existing housing stock;
3. the government's policy with respect to housing, sustainable building, urban renewal and restructuring.

Following on from this, an overview will be given of the solutions and strategies developed in the Netherlands to upgrade the existing postwar housing stock.

### 1.1 Building volume

The Dutch building volume in the housebuilding sector has increased by an average of 3 per cent over the last 2 years.

**Table 1. Building volume in the housebuilding sector in billions of euros**

| | Year | | | |
|---|---|---|---|---|
| Housebuilding Sector | 1998 | 1999 | 2000 (estimated) | 2005 (forecast) |
| • New development<br>• Repair and refurbishment | 8.4<br>4.6 | 8.7<br>4.7 | 9.0<br>4.8 | 8.4<br>5.5 |
| Total (excluding Dutch VAT) | 13.0 | 13.4 | 13.8 | 14.0 |

Based on forecasts made by the Economic Institute for the Construction Industry, the following trends are expected until 2005:

A slight fall (7%) in the building volume of new houses;

A substantial growth (15%) in the refurbishment and renovation sector.

### 1.2 Quality of existing housing stock

In 2001, there are more than 6.5 million houses in the Netherlands. Approximately 70% consists of single-family housing and the remaining 30% is multi-family housing (Ministry of Housing, Spatial Planning and the Environment, MVROM, 2000b).

The share of single-family houses in the big cities is much lower. Most of the single-family houses consist of terraced houses: about 40% of all single-family houses are in the middle of a row of terraced houses, whereas nearly 25% are on the corner of a row of houses. The remaining 35% is equally divided between detached and semi-detached houses. Most of the detached houses are prewar. More than 70% of all houses in the Netherlands have a garden; a quarter of the houses have a garage.

On 1 January 1999, 23% of the total housing stock was prewar, 12% was built between 1945 and 1959, 18% between 1960 and 1970, 19% between 1971 and 1980, 17% between 1981 and 1990, and 11% was built after 1991. There are large differences between the provinces. In Amsterdam, for example, half of the housing stock is prewar.

Nearly half of the houses in the Netherlands are rented (48 per cent) and the rest are privately owned (52 per cent). If this trend-like development continues, 58% of the housing stock will be owner-occupied in 2010. The government's policy is that about 65% of the housing stock will consist of owner-occupied housing in 2010 (MVROM, 2000c). In order to achieve this, rented housing will have to be sold on a large scale (including 500,000 rented houses owned by housing associations). The feasibility and desirability of this is being questioned by the market parties.

Most of the rented housing is owned by housing associations (approx. 2.3 million). The rest (0.8 million) is owned by private and institutional landlords.

It is expected that approx. 80,000 additional houses per year will be built in the period 2000-2004 and approx. 70,000 new houses per year will be built in 2005-2010 (2/3 of which will be on designated expansion locations). In addition, approx. 13,000 houses per year are removed from the housing stock due to demolition (with a further approx. 8,000 houses per year being removed due to joining houses together).

Quite a lot of information is available about the quality of the housing stock in the Netherlands. The main source of this is a national survey carried out by the government on the functional and technical quality of the whole housing stock. The so-called 'Kwalitatieve Woning Registratie' or KWR (Qualitative Housing Registration). This is expressed in terms of the extent of the backlog in housing quality.

The last time this backlog in housing quality was measured was in the period 1994-1996 when it amounted to more than 18 billion guilders (average of approx. 3,200 euros per house). The subsidized rented housing sector relatively speaking has the lowest backlog in housing quality followed by the privately owned housing sector; the private rented housing sector came out worst relatively speaking (MVROM, 1997).

The KWR indicates that thermal insulation in the existing housing stock still leaves much to be desired. In 1994-1996, 57 per cent of houses had double glazing (full or partial), 50% had roof insulation,

41% wall insulation and 22% floor insulation.

In the last decades, the average number of people in a household has dropped to slightly more than 2 people. The available 'living space' has increased to 40 $m^2$ per person.

There is a need for larger houses (more square metres and cubic metres) with higher quality facilities and more spacious building plots (the dilemma in the Netherlands, is the lack of space).

Research shows that nearly 1.1 million houses are not easily accessible and/or easy to reach (dilemma: increasing ageing of the population). It is possible to make about half of these houses accessible by taking reasonably simple structural measures. The other half requires extensive and therefore costly measures to be taken.

Sustainable building gains a foothold in new developments and renovation. However, this is often restricted to a package of cost-neutral measures. A point requiring attention is that the willingness to pay more for sustainability is declining. In the rented sector, 81 per cent of tenants are not prepared to pay more for measures aimed at sustainability. It is slightly better in the owner-occupied housing sector, but even so 67% is not prepared to pay more (MVROM, 2000b).

## 1.3 Government policy

### 1.3.1 Policy Document on Housing

In 2000, the government published a Policy Document (MVROM, 2000c)with a vision on the future of housebuilding. This is based on three principles:

1. more freedom of choice for the housing consumer
2. more attention to social values;
3. more government involvement and a controlled market mechanism.

The policy document focuses attention on five key themes, namely: 1) increase say about housing and the housing environment, 2) create opportunities for people in vulnerable positions, 3) promote tailor-made housing and care, 4) improve the urban quality of living and 5) facilitate housing requirements for green areas.

It is assumed that these themes will be translated into a Housing Act in the course of time. An act that provides for the position and legal certainty of residents, the tasks, powers, responsibilities and liability of governments, sector organizations and housing associations.

The Policy Document wants to promote private houseownership. The aim is to sell 700,000 rented houses in the next 10 years (including 500,000 houses owned by housing associations). Incidentally, there is a lot of criticism from outside about (the feasibility of) this objective. A subsidized house purchase scheme has been in place since 1 January 2001.

The Policy Document wants residents to be much more involved in the municipal housing policy. This does not only mean being involved in the transformation of existing urban districts and areas. But residents should also be more involved in new developments. This is under the direction of the municipalities.

The aim is to have a third of the number of new houses built under a privately commissioned contract starting from 2005. Such a percentage has already been achieved abroad, for example Germany, France and Belgium (MVROM, 2000c). The market parties are opting more for a combination of privately commissioned building contracts and consumer-oriented property development.

Building regulations and requirements regarding the external appearance of buildings will have to be simplified and procedures must be completed more quickly and cheaply.

The Housing Policy Document states that the policy implemented by the government with respect to sustainable building and maintenance should be continued further into the future and that market parties should be given incentives.

The Housing Policy Document gives the following numbers as the city-centre building undertaking for the next ten years:

| | | |
|---|---|---|
| • New building in cities | | 600,000 |
| • Demolition in cities | | 225,000 |
| • Combinations | (160,000) | 80,000 |
| • Restructuring | | 330,000 |

The parties involved subscribe to a substantial shift from building in expansion areas to the redevelopment of city-centre areas. There are different insights into the expected demolition. Depending on the position in the building process, the emphasis is put on maintaining the existing spatial qualities (advocates of selective demolition) or on the complexity and high costs of renovation (advocates of radical demolition).

The Dutch Central Planning Office estimates the total investment in the so-called physical cornerstone of the urban renewal policy implemented by the 25 large cities in the period 1999 - 2005 at nearly 27 billion euros.

### 1.3.2 National programme for Sustainable Building 2000-2004

The Sustainable Building Programme for 2000-2004 focuses on housebuilding, nonresidential and noncivil building and GWW (groundwork, road and hydraulic engineering). The programme includes both new building and existing housing stock. Three special areas requiring attention (‘additional incentives’) will be formulated, namely: sustainable city-centre building, the housing consumer and energy saving (MVROM, 1999).

Sustainability will be included in the regulations by mid-2002. The development of a water performance standard, a radiation performance standard and a material-related environmental profile of a building are currently being worked on.

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One of the basic principles of the policy document is to increase the sustainability of the existing housing stock (for example sustainable use of raw materials and building materials and reducing the consumption of energy). However, the requirements for sustainability with which the existing stock of buildings and houses must comply have still not been set. But on the other hand for example sustainable management is being encouraged by means of specific subsidies, experiments, research and transfer of knowledge.

Housing associations are particularly expected to make a substantial contribution (7 billion euros) to investments in sustainability in the period until 2010 (approx. 4.5 billion of which is for energy-saving measures).

### 1.3.3 Integration and Urban Renewal

An Act of Parliament was introduced in the Netherlands in 2000, aimed at Urban Renewal (MVROM 2000d). This Act mainly concerns the restructuring of city-centre areas. It is an extensive undertaking, which by investing in physical measures contributes to the design of the social and economic cornerstones of urban renewal (figure 1).

The physical measures relate to (replacement) new developments and renovation of houses, nonresidential and noncivil buildings and industrial estates, residential environment, large-scale green areas and environmental quality. In practice, ‘Restructuring’, ‘Urban renewal’ and ‘Urban policy’ appear to overlap each other quite happily. The distinction, defined in the Act, is that the line of approach for restructuring is in the *physical* measures; and in the urban policy the emphasis is on integrating the physical, social and economic components. Physical measures which contribute to solving social and economic problems. Urban renewal is the integrated result in which the investments relate to the physical measures.

The interrelationship between the three cornerstones of urban renewal are represented in the following diagram figure 1:

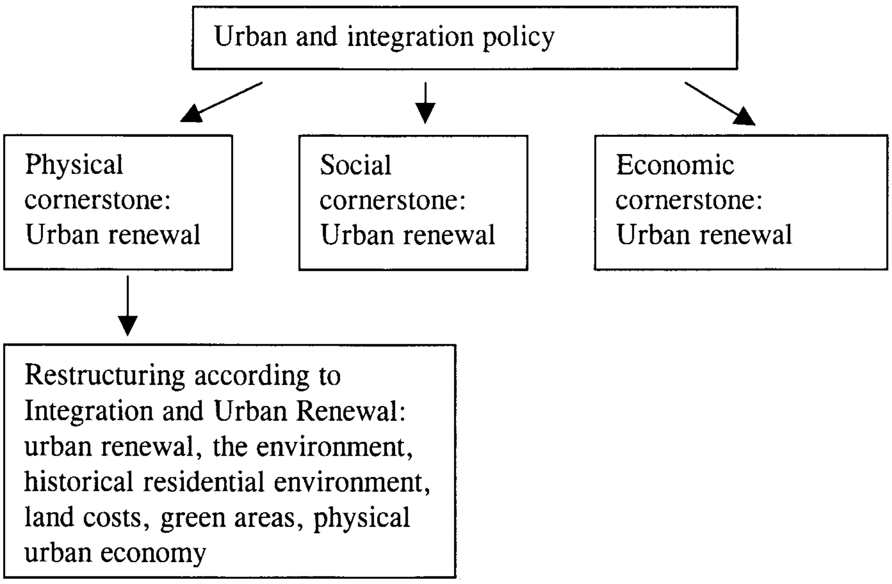


**Figure 1.**

## 2 SOLUTIONS AND STRATEGIES TO UPGRADE THE EXISTING HOUSING STOCK.

### 2.1 Market situation

The housebuilding market has developed from a supply market into a demand market. On the one hand there are houses becoming vacant which are difficult to let or sell in unattractive districts. On the other hand, there is a tight market for spacious, good quality existing houses and new houses.

In general terms, housing consumers are demanding better quality and larger floor areas. In addition, there is a clear need for greater flexibility and adaptability.

The Netherlands is on the eve of extensive measures being taken to the housing stock. This concerns the large-scale redevelopment and restructuring particularly of residential areas built after 1945. This means flats accessed from a common entrance hall, blocks of flats, single-family houses and high-rise flats. This is a substantial part of the housing stock.

More than 2.5 million houses were built in the Netherlands between 1945 and 1975. Housing associations own 1.3 million of these houses, with almost 600,000 in the form of blocks of flats. Most of these houses are three- and four-room flats at the cheaper end of the rented sector.

Despite the efforts made by many housing associations in the 1980's and 1990's, the market position of these houses has decreased substantially. The main reason for this is that the quality of these houses can hardly be improved and if it could be improved, this would only be after making high, uneconomic investments.

The residential districts built in the period 1945 to 1960 have the following characteristics:

- when designed, no attention was paid to the importance of maintenance and management;
- they were built under state control, with the single aim of building houses quickly and cheaply;
- urban planning design was based on the separation of functions (CIAM);
- a lot of shared public space, especially in the form of public parks and gardens;
- standard floor plans were primarily used for all houses;
- large-scale practically identical building plans carried out by one contractor with the aid of prefabricated building systems.

The main problems these districts now have are that:

- houses no longer meet current statutory, functional and technical requirements;
- amenities in the district and housing stock no longer meet the requirements of the residents;
- the composition of the residents has slipped down to the level of people with the lowest income. Eighty to ninety per cent of these houses are rented to ethnic groups.

The problems in these postwar districts in relation to the aforementioned characteristics require a district-oriented integrated approach.

### 2.2 District-oriented and integrated approach

Social and economic aims cannot be realized at a complex level. A district-oriented approach allowing for physical measures above the scale of the building plans is a precondition. This requires planning that takes into account unbalanced types of ownership, various owners and interests and diverse aims. Various budgets and processes will have to be brought together, often in a lengthy process. The interests of sitting tenants encourages transparency and intensive consultation at a stage as early as possible in the planning process. This is to prevent preconceived ideas and the fear of large-scale demolition from arising.

A district-oriented approach also provides room to improve urban planning quality, the main factor for the future value of housing complexes. For a lot of parties who make planning decisions, the concept of urban planning quality is not very specific, let alone making it capable of being tested.

It goes without saying that the municipality, as the party that draws up long-term development plans, exercises control over the process. In practice, it appears that municipalities are often still insufficiently equipped to direct the complex process of urban renewal.

### 2.3 Tools required for restructuring

The traditional approach of managing the housing stock, especially by housing associations, has three forms:

- regular maintenance and replacement of building components aimed at maintaining the property;
- major repairs aimed at qualitative home improvements;
- replacement new building by carrying out massive demolition of houses.

This traditional approach has not proved to be adequate to keep the district as a whole up to a certain standard. Encouraged by government policy, progressive housing associations and residents' associations, a range of strategies and solutions has been added to the existing traditional approach over the last 5 years.

The strategy employed in addition to this has the following broad elements:

- using the abundant public space especially the green areas to create more private spaces, such as allotments, play areas and private gardens next to stacked housing;
- breaking through the separation of functions by combining residential and business areas and encouraging urban living in the district;
- restructuring the existing unbalanced housing supply into a differentiated supply for several target groups, also with combined home and office;
- applying high requirements with respect to urban planning and architecture.

The solutions developed by applying this strategy will now be dealt with briefly, divided into solutions for low-rise buildings and for stacked housing (blocks of flats). The various solutions are illustrated with practical examples.

### 2.4 Strategy for low-rise building

Residential districts built in the 1950's and 1960's are characterized by monotonous rows of the same type of house, only intended to provide housing for families with children. Here and there this is interspersed with terraced houses for elderly people.

The approach used for low-rise building focuses on:

- qualitative improvement of the possibilities for use, environmental and energy performance and the comfort of the house.
- extending the floor area by providing residents with a package of options;
- breaking through the architectural monotony by enlarging the house combined with a series of external wall variations.

### 2.5 Low-rise building pilot project (Hoogeveen, the Netherlands)

The Municipal Council of Hoogeveen wants to demolish the whole of the Wolfsbos district of Hoogeveen built in 1967 to 1969 because of the poor quality and the difficulty of letting the houses. The residents only want it to be renovated. The owner (housing association) of the houses has decided, based on an analysis which included the urban planning quality of the district and differences in quality between the various types of houses, to take a combined approach of limited demolition (14%) and replacement new building with a differentiated renovation of 76% of the houses. The new houses are especially aimed at attracting new higher-income target groups to the district.

**Figure 2 Various ways of extending houses and types of architecture for the houses to be renovated**

Extending the houses firstly carries out the requirements of the sitting tenants and secondly is used architecturally to reinforce the spatial picture of the district. (figure 2)

The main measures with respect to sustainability are:

- using durable materials;
- separate collection of rainwater for use in the district and direct infiltration into the soil;
- replacing collective district heating with individual heating systems making use of solar collectors.

An important strategy which operated as a control mechanism for urban planning design is that new owner-occupied houses faced onto an inner courtyard of the plan and not onto the green outer edges.

The argument here is that a good balance is created between the lettability of the houses to be renovated on the green periphery and the owner-occupied houses to be built.

### 2.6 Strategy for stacked building

Most of the flats dating from the period 1945 to 1960 were built in linear rows in an open parcelization. The housing complexes usually have 3 to 5 storeys.

The ground floor is in direct contact with the public space (parking and green areas) and do not have hardly any living space but nearly always just storage space.

On average, the houses have three to four rooms with a floor area of 60 to 70 m².

This floor area can now only be let to one or two households with a low to average income.

In order to be able to retain these housing complexes, they must be made socially and economically attractive again. The main condition is that the unbalanced supply is widened into a differentiated supply aimed at three target groups:

- elderly people with low incomes;
- large households with low incomes (especially ethnic groups);
- households consisting of 1 or 2 people and families with high incomes.

In order to be able to realize such a mix within an existing district without having to carry out large-scale demolition of houses, a range of solutions have been experimented with over the last few years.

All these solutions are based in one way or another on transforming the existing complex.

The various sorts of solutions applied in practice are:

a) vertically combining two houses one built above the other into one new house;

b) horizontally combining two adjacent houses into one new house;

c) adding an additional floor on the roof (termed ‘optoppen’, topping up);

d) replacing the storage spaces on the ground floor with large houses consisting of two floors. This means that the small house above the storage space no longer exists. These houses also have their own garden (termed ‘uitplinten’, changing the function of ground floor);

e) partial demolition of upper floors in such a way that spacious single-family houses can be created in the remaining lower space (termed ‘aftoppen’, topping);

A completely new form which is still in the development stage is based on a partial adaptation of the existing shell of a complex. The basic idea here is that for each house one solid loadbearing wall is replaced by an industrially manufactured portal frame with the result that a much more spacious and flexible layout is created for installing a differentiated fitting-out package into houses with a high degree of flexibility. This approach is discussed in the paper by Hendriks, Van Nunen and Rutten entitled ‘Flexible Knocking Through Project: Methodology and Design’.

## 3 VERTICAL COMBINATION PILOT PROJECT

The three-storey flats accessible from a common entrance hall (average of 44 m2) in this complex which was built in 1953 have been converted into individual houses. The floor plans show how both The floor plans show how both narrow and broad types of new houses have been designed. Part of the top floor has been demolished to create a roof garden. (Figure 3)

Old flat: narrow and broad | New house: narrow type | New house: broad type

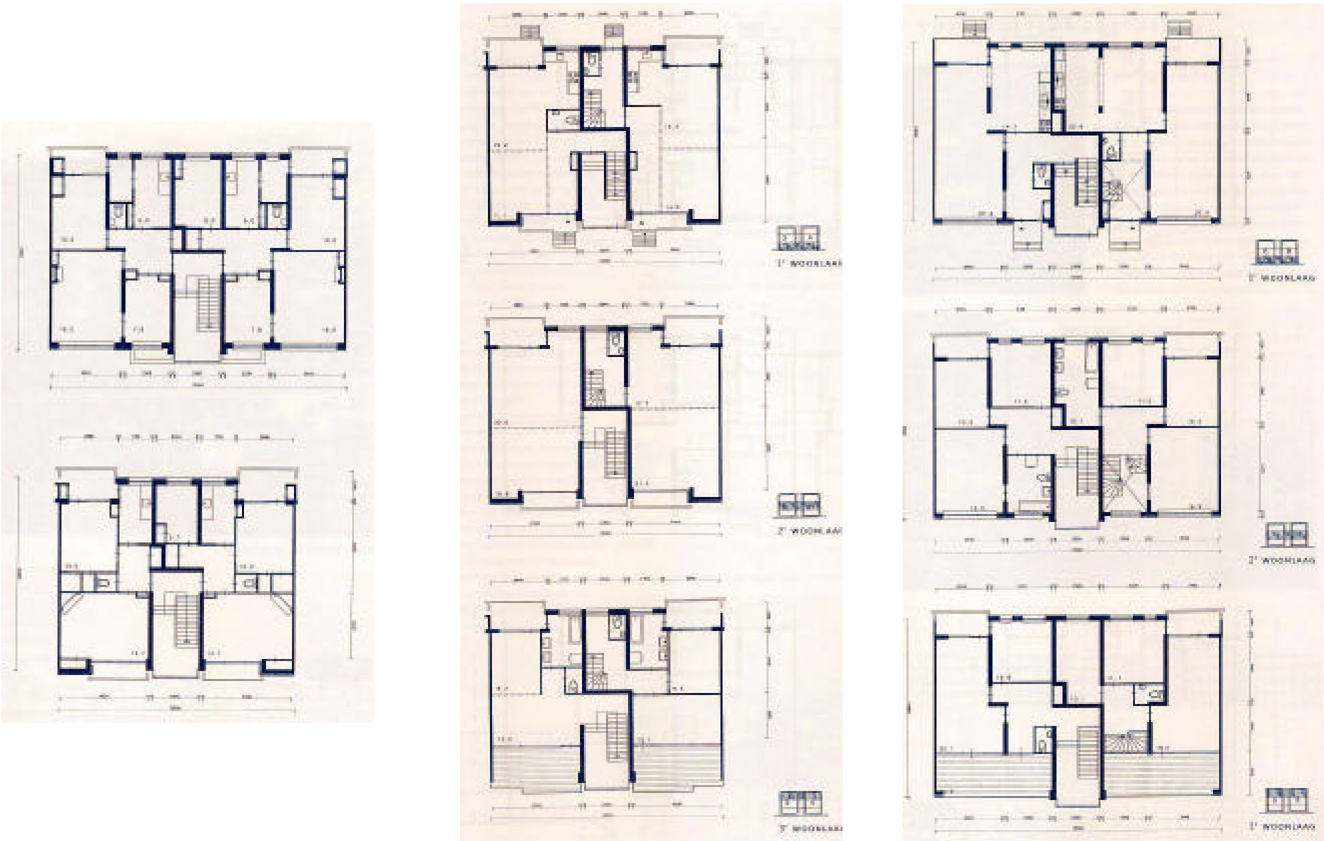

**Figure 3**

## 4 HORIZONTAL AND VERTICAL COMBINATION PILOT PROJECT

This example concerns an extensive urban renewal project in one of the large southern garden cities of Rotterdam, the Pendrecht district. A district with 6,000 houses built in the 1950's and 1960's. Ninety per cent of the district consists of subsidized rented housing in stacked complexes.

Figure 4.In order to prevent the district from deteriorating even further and vacancy from arising there due to the poor quality of the houses and the unbalanced supply, the Rotterdam housing association has taken the initiative for a well-thought out combination of maintenance, restructuring and new development.

The unbalanced supply of only cheap flats has been redistributed into 70% upgraded cheap flats and 30% expensive houses.

The restructuring consists of the horizontal and vertical combination of small flats into large houses for families. The process is controlled by the participation and requirements of the residents.

### 4.1 From three to two

The strategy with respect to the blocks of flats accessed from a common entrance hall includes vertical combination, major repairs and home improvements. Two out of each block of five flats are reserved for combination. Part of the housing units on the second floor is added to those on the first floor and part to those on the third floor. Major repairs consist of a uniform approach with respect to external walls and pipework, and an approach with respect to the fitting-out work which is at the resident's choice. The options available to the residents range from a modest upgrading to a completely new house. The composition of each fitting-out package is arrived at in close cooperation with the residents. (Figure 5)

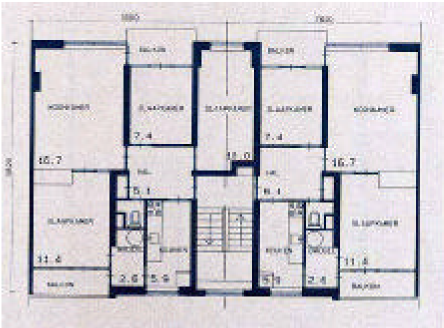

**Figure 4. Existing floor plan of flats accessed from a common entrance hall**

## 5 TOPPING UP PILOT PROJECT

The Municipality of Schiedam has drawn up an integrated plan for urban renewal for the Nieuwland district (6,700 houses) built between 1950 and 1960. The approach, besides traditional solutions such as major repairs and home improvements also consists of a series of innovative restructuring proposals such as:

- in the case of blocks of flats, combining the built-in lock-up garages and the flats above into spacious family houses with access on the ground floor;
- extending houses by building conservatories and roof gardens;
- making houses suitable for elderly people;
- adding housing for the elderly on the roof of blocks of flats accessed from a common entrance hall (topping up).

As part of the plan, the municipality has applied the so-called topping up in order to meet the need for affordable housing for the elderly. Topping up is realized by means of a timber-frame construction to enable the building work to be carried out quickly (completion after 16 weeks) and so the maximum possible increase in weight of 10% is not exceeded.

This technique of topping up has also been used in other cities over the last few years. The Foundation for Building Research, together with the Steering Group on Experiments in Public Housing, has evaluated the various projects.

The following lessons have been learned from this evaluation.

Topping up makes it possible to redifferentiate two target groups:

- *elderly people with low incomes.* A condition here is fitting an additional lift. This is easily feasible economically in gallery flats. But is much less so in flats accessed from a common entrance hall. In the latter case, various solutions are possible such as converting flats accessed from a common entrance hall into gallery flats or adding a lift to the flat accessed from a common entrance hall which then serves a gallery with the roof top housing units.
- *households with higher incomes.* Topping up is used here to add luxury housing such as penthouses. This solution often makes use of light prefabricated steel structures.

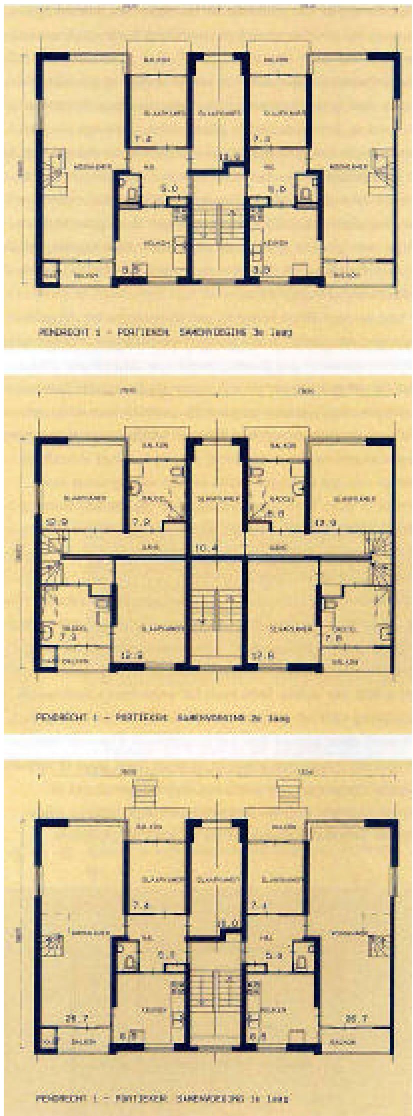

**Figure 5. Vertical combinations on the 1st, 2nd and 3rd floors**

## 6 GROUND FLOOR PILOT PROJECT

So-called 'uitplinten', changing the function of the ground floor is another example of a type of innovative restructuring with the aim of redifferentiating the existing housing stock in the postwar districts.

The definition of 'uitplinten' is: the realization of space for housing, working and facilities on the bottom floor or floors of a block of flats. The existing storage space and lock-up garages are partially removed.

The aim of this type of redifferentiation is principally to increase the urban quality of the ground floor, in addition to being designated for larger houses (by combining with the flat above). As a result, the cohesion and quality of life in the district are strengthened. In addition, by converting public green areas into private gardens and creating individual entrances (houses and workspaces), social control and the sense of space are very much improved.

With respect to the line of approach, the same as the other examples, it can be stated that this form of urban renewal considerably reduces the impact on the environment by making use of the existing buildings and infrastructure so that the life of the complexes is drastically lengthened. These buildings are assumed to have a life of from 30 to 50 years.

## 7 CONCLUSION

A strategy for the redevelopment of residential districts built in the period between 1950 and 1965 requires solutions which are based on:

- keeping the existing spatial structure, especially the green areas and buildings;
- redifferentiating the housing stock by using a range of architectural restructuring concepts such as: vertical and horizontal combination, 'optoppen'(topping up, building roof-top housing units, 'uitplinten'(changing the function of the ground floor), 'aftoppen' (topping, removing one of more top floors) and knocking through (Hendriks, Van Nunen and Rutten);
- applying sustainable improvement and fitting-out techniques which are aimed at realizing savings in energy and water, flexibility and the use of environmentally friendly materials.

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# Project Flexible Breakthrough Methodology And Design

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**Summary: In the Netherlands housing corporations hold 1,3 million houses, of which almost 600.000 are realised in buildings with three- and four-room apartments in the low cost renting sector. They were built between 1945 and 1975. The market position of these houses has been deteriorating substantially, although they are located in very pleasant quarters of the large and medium-sized cities. After renovation the houses remain too small, poorly equipped and noisy. For this reason a research program was started. The objective of this program is to - more or less - reconstruct the apartment buildings, at the same time using, as much as possible, IFD-technology. IFD stands for: Industrial, Flexible and Demountable. This resulted in the concept 'Flexible Breakthrough'. The basic principle of the project is to completely remove (demolish) one of the four bearing walls in each apartment and replace this wall by a steel supporting frame. The resulting much larger space is to be redesigned with IFD-technology.**

**The paper describes the systematic methodology to analyse the possible solutions and the design of the Flexible Breakthrough concept. This concept provides maximal industrialisation, flexibility and demountability within the given constraint of retaining the major part of the existing apartment building.**

**Keywords: IFD-technology, apartments, existing buildings, partial demolition**

## 1 INTRODUCTION

Between 1945 and 1975 more than 2,5 million houses have been built in the Netherlands. Housing corporations hold 1,3 million of these, with almost 600.000 realised in apartment buildings. Most of these houses are three- and four-room apartments in the low-cost renting sector.

Despite the efforts of many housing corporations in the 80's and 90's the market position of these houses has decreased substantially. The most important reason for this is that the quality of these houses hardly can be improved and if so, only by high, uneconomic investments. The renovated houses remain too small, poorly equipped and noisy. On the other hand, many of these apartment buildings are located in very pleasant quarters of the large and medium-sized cities, which makes them attractive for rehabilitation.

For this reason, Stichting Bouwresearch (foundation Building Research) in co-operation with housing corporation 'het Oosten', one of the largest Dutch corporations in Amsterdam and the Eindhoven University of Technology, initiated a research program. The objective of this program is to **-** more or less **-** reconstruct the apartment buildings, at the same time using, as much as possible, IFD-technology**.** IFD stands for: Industrial, Flexible and Demountable. This resulted in the project 'Flexible Breakthrough'. The basic principle of the project is to completely remove (demolish) one of the four bearing walls in each apartment and replace this wall by a steel supporting frame. The resulting much larger space is to be redesigned with IFD-technology. The advantages of this approach could be:

1. Substantial reduction of waste, due to less demolition and application of IFD-technology.
2. Better possibilities for the improvement of the houses with respect to acoustical properties and quality and flexibility of building services.
3. Complete demolition and new construction would cost about € 27.000, **-** more per house.
4. Faster availability of apartments for rent.

## 2 IFD TECHNOLOGY

Important aspects of IFD technology are (Hendriks & Jacobs 1999):

- industrial construction: prefabrication, which means also less waste with the actual production, often production recycling is feasible;
- no waste on the building site, which is a boundary condition;
- construction becomes assembling: completely dry building method, which is also a boundary condition;
- flexible also means "changeable" during the course of life of the building, so there is also less waste;
- flexible in the design phase means for example that the developer of the building can wait until the last moment with final decisions about the lay-out of floors;
- demountable also means that reuse or at least recycling is possible;
- perhaps IFD technology can mean: less construction (in general).

### 2.1 Design criteria

For the design on changeability the following criteria could be used (Hendriks & Vingerling 2000):

*Integration and independence of disciplines:*

- supporting structure
- installations
- building envelope
- interior finishing

*Completely dry construction method, which means:*

- no in situ concrete
- no mortar joints
- no screed floors
- no plaster
- no sealant
- no in situ polyurethane

*Perfect modular measuring, which means:*

- extreme attention to drawing work
- prototype testing on:
  - –mountability
  - –functionality
  - –demountability
- quality system drawing work
- assembly instructions

*Changeability on all aspects:*

- supporting structure (limited)
- installation (practically unlimited)
- building envelope (limited and modular)
- interior finishing (practically unlimited and modular)

Several of these principles are similar to those of the 'Open Building' concept. An important difference of the IFD-principles however is that there is no distinction between 'base building' and 'fit-out'. On the contrary, as mentioned, aspects like changeability, integration and mutual independence apply equally to both of them.

Because one of the four bearing walls has to be demolished it was not possible to apply all the principles of IFD-technology in every detail. The consequence of the removal of such a wall is that the floor slabs also partly must be demolished. After the installation of the new steel-supporting frame the floor slabs have to be reconnected, which must be done with in situ concrete. But for the remaining part IFD-technology can be applied.

## 3 METHODOLOGY

The methodology that has been used by the design team is based on the approach that has been introduced by Rutten (1997). The members of the design team represent the following disciplines: IFD building technology, structural design, building services and architecture. The results of the design teamwork are presented at every relevant stage to a steering committee with representatives of the housing corporations and Stichting Bouwresearch. First so-called system development areas have been selected. Evaluation points relate to critical details and other aspects of the design that need to be analysed. For every evaluation point a maximum of three alternatives has been determined. The evaluation of these alternatives is a balanced multi-criteria selection process to not only attain a high degree of industrialisation during construction, extreme flexibility during use of the building and demountability at the demolishing stage of the building project, but also a high degree of sustainability and personal comfort. This will be demonstrated by two examples.

### 3.1 System development areas

The following system development areas have been selected:

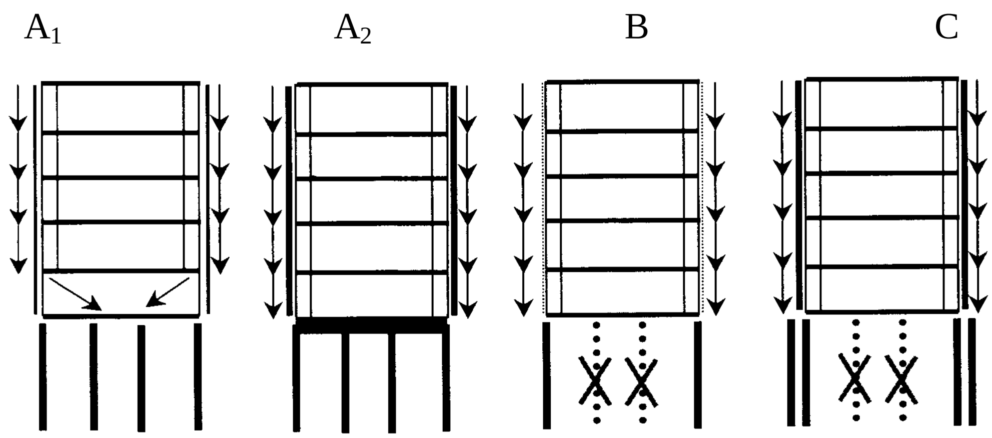
1. Foundation
2. Position of supporting beam
3. Sound proofing
4. Demolition
5. Construction and stability
6. Façade
7. Building services
8. Execution

For every evaluation point also the relation with other points have been given.

### 3.2 Example: Foundation

Removal of the bearing wall means a lower load to the foundation of approximately 80 tons. On the other hand the distribution of the load also is changed. The consequences are analysed in table 1.

**Table 1. Analysis of foundation***




| | | | | |
|---|---|---|---|---|
| **Alternative** | Removal of bearing wall, except at the (subterranean) ground floor (mostly only for storage). | Removal of bearing wall on all floors and installation of heavy foundation beam. | Removal of bearing wall on all floors with light façade. Maintaining existing piles. | Removal of bearing wall on all floors and extra piles at the exterior. |
| **Advantages** | All piles contribute. Freedom in choice of façade. No need for extra piles. | All piles contribute. Freedom in choice of façade. No need for extra piles. Flexible layout on *all* floors. | Due to removal of bearing wall and use of light façade no need for extra piles | Freedom in choice of façade. Weight of construction is less critical. |
| **Disadvantages** | No flexible layout at ground floor. | Installation of large beam under floor: space consuming and costly. | Only light façade is possible. | Extra piles required. |
| **Observations** | Because of storage function of ground floor, flexibility is less needed. Ground floor wall is mainly made of poured in | Can be combined with alternative A1 in cases with houses at ground floor level. | Feasibility must be assessed for every separate project, accounting for bearing capacity piles, soil condition, etc. | Alternatives B and C are identical, apart from the need for extra piles, which |

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| | | | | |
|---|---|---|---|---|
| | place concrete: very solid construction. | | | depends on bearing capacity. |

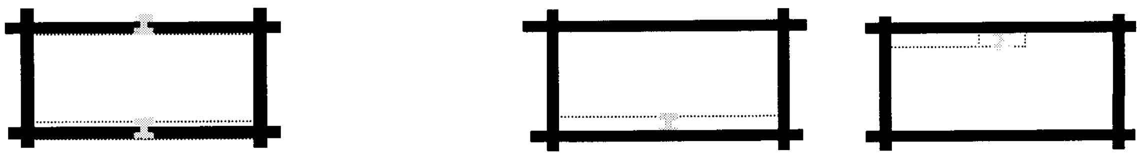
***Relation with façade and construction and stability***

### 3.3 Example: Position of supporting beam

Furthermore the analysis of the position of the supporting beam is given. This is done because this position is crucial in the whole concept. Table 2 illustrates the alternatives as well as the considerations for the final selection, which is alternative A.

**Table 2. Analysis of supporting beam position***

A B C



| | | | |
|---|---|---|---|
| **Alternative** | Beam in the floor slab, with raised floor and finishing ceiling | Beam on top of the floor, with raised floor | Beam under the floor, with either a locally or completely lowered ceiling |
| **Advantage** | Minimal loss in height of room (2,43 m free).<br>Floor is also flexible space for pipes and ducts<br>Optimal flexibility<br>Demolition is possible through the roof<br>Acoustical improvement of the floor<br>Electrical wiring in floor, but also in ceiling | Floor is also flexible space for pipes and ducts<br>Optimal flexibility<br>Acoustical improvement of the floor | Extra floor is not necessary<br>With locally lowered ceiling remaining height of room is 2,59 m |
| **Disadvantage** | Thresholds and doors must be adapted | Free height of room is not more than 2,27 m<br>Thresholds and doors must be adapted | With completely lowered ceiling free height of room is 2,28 m<br>Sewers of upstairs neighbours through apartment<br>No acoustical improvement of the floor |

| | | | |
|---|---|---|---|
| | | | In case of locally lowered ceiling connection of toilets etc. only near beam Limited flexibility |
| **Observations** | Lowest loss in roomheight, with acoustical improvement and finishing possibilities Piping and ducting for building services can be integrated in the supporting frame | A beam is also required on the roof | |

* *Relations with demolition and building services*

Every system development area has been analysed in a similar way. Figures 1 through 4 give an impression of the demolition and installation procedure.

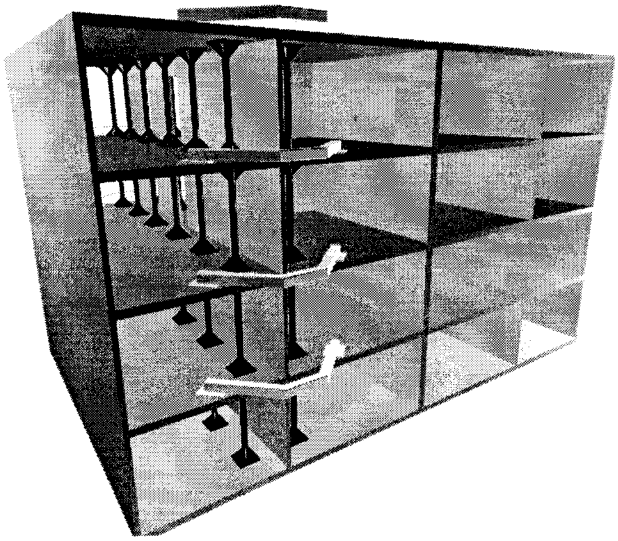


**Figure 1. Positioning of temporary supports.**

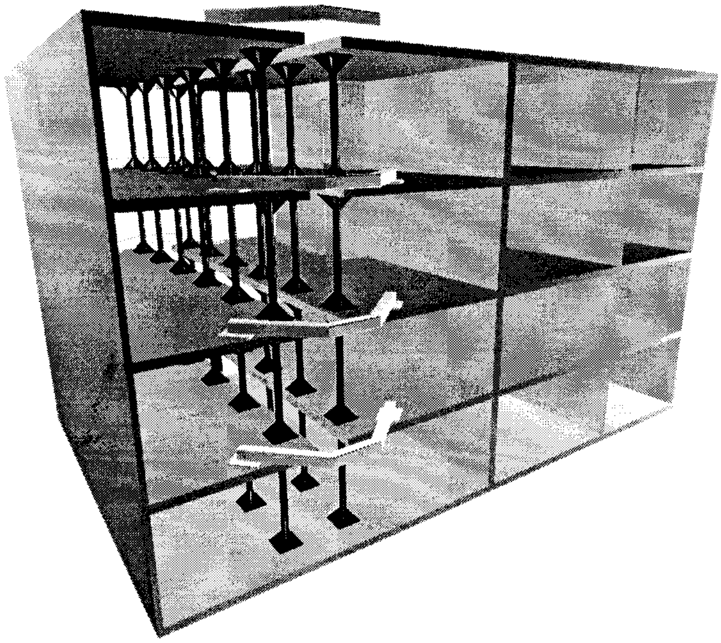


**Figure 2. Removal of the bearing walls.**

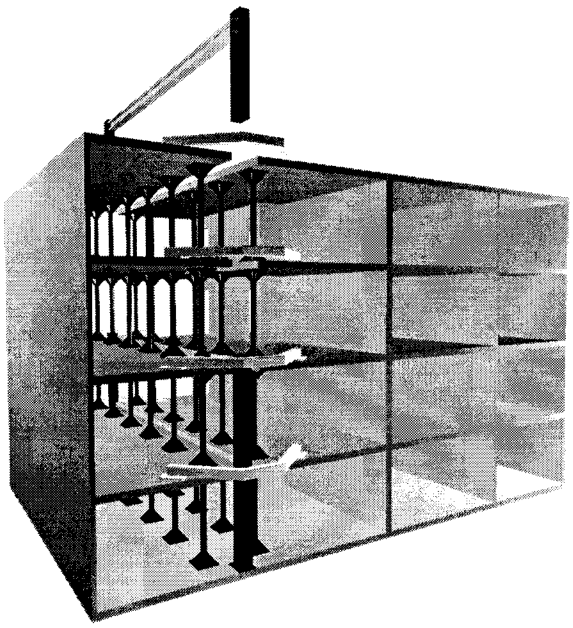


**Figure 3. Installation of the integrated supporting frames.**

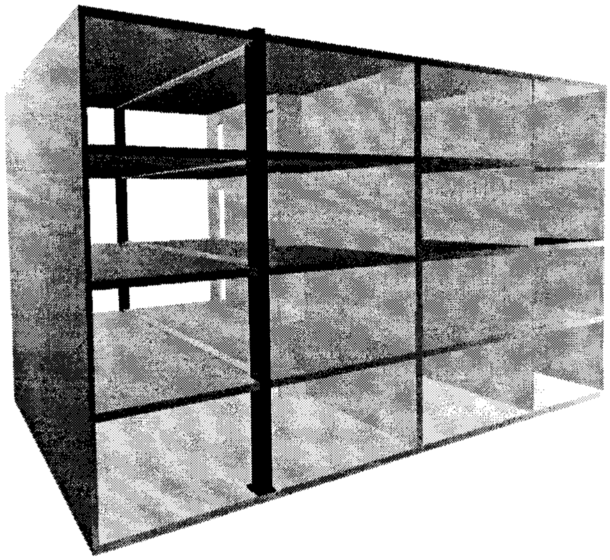


**Figure 4. Completed structure.**

## 4 FURTHER RESEARCH

During the progress of the study two more housing corporations joined the project: Far West in Amsterdam and HaagWonen in The Hague. They have contributed substantially in the evaluations. The positive conclusions of the feasibility study have resulted in a consecutive study in which they participate together with housing corporation ‘het Oosten’ and Stichting Bouwresearch (Foundation Building Research). The further study proved that the crucial element of the project is the integration of building services. Apart of the mentioned analysis also a mock-up was made (1:25) to study the arrangement of distribution systems of the building services. Figure 5 shows a picture of the mock-up. The final design of detailing around the supporting steel column is given in figure 6. The supporting frames will be prefabricated completely with all the building services.

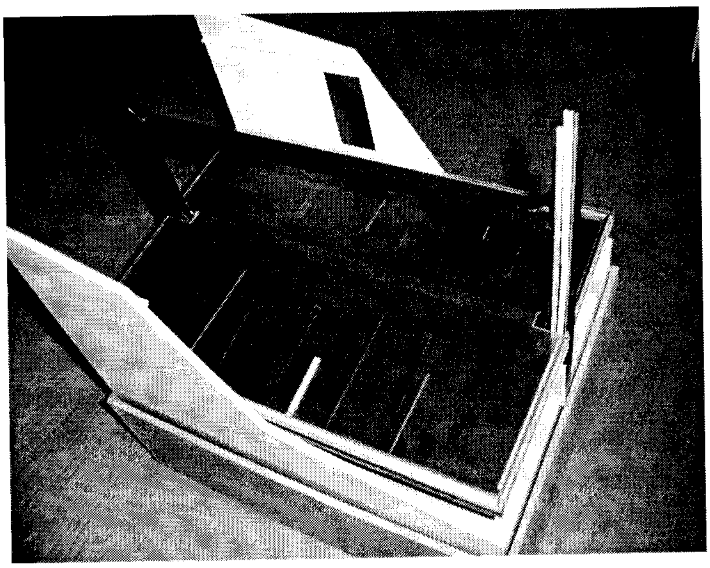


**Figure 5. Mock-up scale 1:25.**

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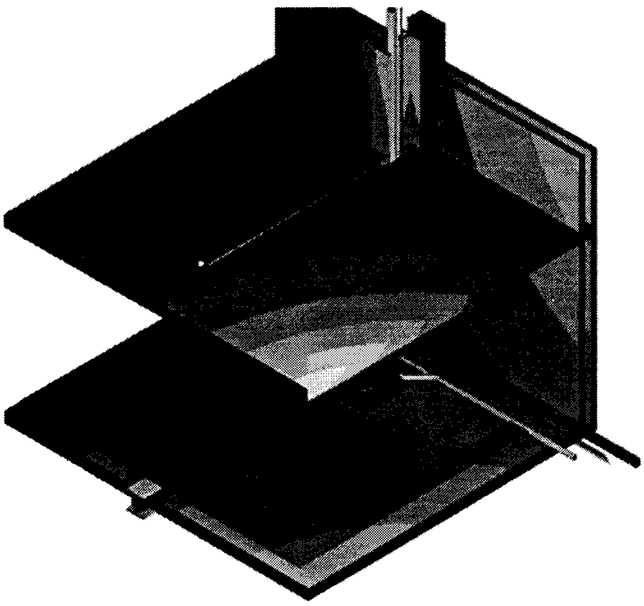

**Figure 6. Integration of building services**

The essential element of this study is the prototype testing on a real project. First part of the testing is the execution of the demolition and installation part of the ‘Flexible Breakthrough’ concept. This will be done on an apartment building that already has been designated for demolition. After evaluation of the results the concept will be tested on location where the inhabitants will return. The results of this study will be reported in due course.

## 5 RESULTS AND CONCLUSIONS

From the research in the first phase of the project can be concluded that the ‘Flexible Breakthrough’ concept is very feasible. The balanced multi-criteria selection process not only attains a high degree of industrialisation during construction, extreme flexibility during use of the building and demountability at the demolishing stage of the building project, but also a high degree of sustainability and personal comfort.

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# Integrating Building Components In Roof System Design For Long Term Roof System Performance

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**Summary: As an architect who has specialized in roof system design for the past 17 years, the author has observed that most roof leaks and the resultant interior damage are the result of a poor understanding and detailing of how building components are integrated into the roof system. Roof system leaks are the No. 1 concern of both architects and building owners, especially now that *sick building syndrome* concerns are prevalent in the U.S.A. construction industry. In order to achieve long-term performance of any roof system, comprehensive detailing of the building components and roof system interfaces are required. This paper will review the importance of including the various components that interface with the roof in the roof design. With ease of maintenance in mind, details that reflect this concept and have long-term performance and case studies of *in situ* performance will be presented. Practical guidelines on how to involve designers of other building systems such as plumbing, structural, and mechanical will also be reviewed.**

**KEYWORDS: HVAC Equipment, Roof Penetrations, Masonry, IT Antennae, Roof System Design**

## 1 INTRODUCTION

To be successful roof system design must take a holistic approach. A building design process that does not take into account all roof-to-building-component interfaces will fail to achieve optimal roof service life. Roof designers must become familiar with how buildings work and how the various building systems and roof system must work in concert to achieve optimal service life. This paper will review the design and coordination processes, and details required for continuous success. The reader must accept the concept that roof system design is on equal status with other systems' design, such as HVAC, structural, and plumbing, and that roof system design requires the same dedication, hard work, and coordination--a concept that clearly 'flies in the face' of the building design process currently being practiced in the U.S.A.

## 2 CURRENT DESIGN PRACTICES

While the American Institute of Architects has recommendations and checklists regarding the design process and the preparation of contract documents, there is no industry or national standard regarding this process. Design creativity by its nature is most often a solitary affair. Building design is often created in a vacuum (the architect's office) with little if any input from consultants whose components will impinge upon the design. Often building systems (structural, HVAC, plumbing, etc.) are requested to fit into a designed building rather than evolve with the building's design. Those buildings that evolve through the participation of building system designers, including roof system design, have a greater chance of long-term functional success for the building user as well as achieving design excellence.

The concept of issuing a set of construction documents for bidding with a properly designed roof system with well thought out details and specifications is unfortunately not often the case. Much too often architects rely on manufacturers' standard details that are seldom germane to the specific building conditions. The results are as expected with moisture-related concerns being the number one cause of lawsuits against architects. The concept of incorporating building system and building components into roof system design and detailing is almost unheard of.

While the ISO 15686 standards on service life planning are laudable, this author, working in a large international firm and having contact with all the major Chicago-based architectural firms, knows of not one architect who is cognizant of ISO 15686; consequently practicing compliance with this standard is virtually nonexistent. This presents the challenge to the research community to enlighten the profession, and is beyond the scope of this paper.

## 3 DESIGN CONSIDERATIONS

Building design is composed of three phases prior to the onset of the preparation of construction drawings: preliminary design, schematic design, and design development. For the purposes of this paper, it is assumed that the roof system designer is cognizant of the climatic, environmental, and user demands that the roof must endure. It is also assumed that the appropriate roof covering has been selected. The process of roof covering selection is beyond the scope of this paper, but it is sufficient to say that this selection process is usually finalized in the design development phase.

It is during the schematic building design phase when the building components are being coordinated that the roof system designer should begin to investigate just how the various building systems affect the roof system. Building components that need to be considered because they impinge on the roof are

- Building enclosures and architectural elements
  Parapets

  Walls
- Structural components
- Plumbing components
- HVAC components
- Maintenance concerns
- Information and cellular technology hardware
- Miscellaneous building elements

Once the building systems that will impinge on the roof are identified, those components that directly interface with the roof need to be identified. Components such as the following need to be recognized:

- Building Enclosures and Architectural Elements
  - Masonry walls
  - Curtain walls
  - Skylights
- Structural Components
  - Roof deck systems
  - Support of curbs, roof drains, mechanical systems
  - Miscellaneous steel
- Plumbing Components
  - Roof drains
  - Plumbing vents
- HVAC Components
  - Exhaust fans
  - Rooftop units (Rtu's)
  - Condensing units
  - Rooftop piping, ductwork, electrical
  - Associated piping from rooftop through roof
- Maintenance Concerns
  - Ladders
  - Roof hatches
  - Window washer systems
  - Future reroofing
  - Walkways
- Information and cellular technology hardware
  - Antennae
  - Wiring
  - Rooftop-to-interior transitions

Once all impinging components are identified, roof system design and detailing to successfully accommodate them can begin.

## 4 DESIGN

In designing and detailing building components to interface with roof systems, the following concept is often forgotten, which has led to many a moisture infiltration condition:

Design and detail, where possible, building components to roof system interfaces so that they can perform independently of each other.

Detail drawings which appear later in this paper will show how this concept translates into a workable product.

Building system consultants and those responsible for the building design should be contacted and informed of your design efforts and a coordination meeting arranged. The author finds it beneficial to review drawings prior to the meeting so as to be current with project status. The intent, purpose, and goal of the meeting should be explained. All attendees have a vested interest in the project's success. Impediments to success such as an unwillingness to change, an is-all-this-extra-work-necessary attitude, and lack of respect for the roof system designer often arise,. Diplomatic skills are as important as technical skills at this point in the project; all involved need to agree that this is an integral step of the design process if optimal roof system design is to be achieved and client satisfaction guaranteed.

## 5 DETAILING: CONSTRUCTION DRAWINGS

The author has found it advantageous to prepare comprehensive construction drawings, plans, and details on a large scale (1:4 for details) so as to properly communicate the design intent to bidding contractors. Coordination between the roof plans and details and the impinging system's plans and details is imperative. Roof drain, roof curb, gas piping, and transition details should reflect and coordinate with those details shown on the respective plumbing and HVAC drawings.

As the roof system design is being committed to construction drawings and the plans begin to reflect drainage paths, all the previously identified building components should be shown to scale on the roof plan. Components such as rooftop units and large roof curbs may affect the performance of the tapered drainage system. Conflicts such as this which will affect the roof systems performance need to be brought to the attention of the respective consultant, discussed, and resolved. Can a conflicting element be relocated or rearranged to allow for proper drainage? More often than not when caught early in the construction document phase, modifications such as moving curbs, drains, piping, etc. can be accomplished. If an element can't be moved, perhaps the roof design can be altered. The importance of having a good working relationship between consultants and building designers can immediately be recognized. As drawings proceed toward completion, revisions become more difficult.

Once all components have been located on the roof plan, detailing them proceeds. The roof designer will need to investigate the physical characteristics of the building components with various consultants. What exactly does the air condensing unit curb look like? How is the roof drain integrated into the structure and roofing system? And so on. Cooperation back and forth is required. This process can become laborious. To help alleviate this possibility, the author has prepared a coordination process that greatly reduces time involvement, resulting in greater success.

The author first prepared a comprehensive list of possible HVAC, plumbing, and structural details that if utilized would impinge on the roof system (see Figure 1). Roof system designers review this list as construction documents begin; all details that are relevant are checked.

Figure 1. A comprehensive list of possible HVAC components that may impinge on a roof system can be reviewed, and those that are appropriate checked using the form below. Relevant details of these components can then be forwarded to the HVAC consultant for review and inclusion in their drawings.

HVAC Consultant Details

Index of Details

Project ____________________________

| Provide detail for info only | Coordination detail to be included in HVAC drawings | HVAC Condition |
|---|---|---|
| ☐ | ☐ | M1 – HVAC Unit Curb Detail – Ballasted EPDM Roof System |
| ☐ | ☐ | M2 – Roof Top Unit Curb Detail – Ballasted EPDM Roof System |
| ☐ | ☐ | M3 – Pipe Support Curb Detail – Ballasted EPDM Roof System |
| ☐ | ☐ | M4 – Flue Roof Curb Detail – Ballasted EPDM Roof System |
| ☐ | ☐ | M5 – Pipe Penetration Curb Detail – Ballasted EPDM Roof System |
| ☐ | ☐ | M6 – HVAC Unit Curb Detail – Fully Adhered EPDM Roof System |
| ☐ | ☐ | M7 – Rooftop Unit Curb Detail – Fully Adhered EPDM Roof System |
| ☐ | ☐ | M8 – Pipe Support Curb Detail – Fully Adhered EPDM Roof System |
| ☐ | ☐ | M9 – Flue Roof Curb Detail – Fully Adhered EPDM Roof System |
| ☐ | ☐ | M10 – Pipe Penetration Curb Detail – Fully Adhered EPDM Roof System |
| ☐ | ☐ | M11 – HVAC Unit Curb Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M12 – Rooftop Unit Curb Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M13 – Pipe Support Curb Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M14 – Flue Roof Curb Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M15 – Pipe Penetration Curb Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M16 – HVAC Unit Curb Detail – Built-Up Roof System |
| ☐ | ☐ | M17 – Rooftop Unit Curb Detail – Built-Up Roof System |
| ☐ | ☐ | M18 – Pipe Support Curb Detail – Built-Up Roof System |
| ☐ | ☐ | M19 – Flue Roof Curb Detail – Built-Up Roof System |
| ☐ | ☐ | M20 – Pipe Penetration Curb Detail – Built-Up Roof System |
| ☐ | ☐ | M21 – HVAC Unit Curb Tie-In Detail – EPDM Roof System |
| ☐ | ☐ | M22 – HVAC Unit Curb Tie-In Detail – PVC Roof System |
| ☐ | ☐ | M23 – HVAC Unit Curb Tie-In Detail – Built-Up Roof System |
| ☐ | ☐ | M24 – HVAC Unit Curb Tie-In Detail – Hypalon CSPE Roof System |
| ☐ | ☐ | M25 – HVAC Unit Curb Tie-In Detail – Modified Bitumen Roof System |
| ☐ | ☐ | M26 – Roof Top Unit Curb Tie-In Detail – EPDM Roof System |
| ☐ | ☐ | M27 – Roof Top Unit Curb Tie-In Detail – PVC Roof System |
| ☐ | ☐ | M28 – Roof Top Unit Curb Tie-In Detail – Built-Up Roof System |
| ☐ | ☐ | M29 – Roof Top Unit Curb Tie-In Detail – Hypalon CSPE Roof System |
| ☐ | ☐ | M30 – Roof Top Unit Curb Tie-In Detail – Modified Bitumen Roof System |

Using this checklist, details for each entry are prepared in a unique method. The best way to explain the method is by example. For instance, in the case of a rooftop exhaust fan, there are four trades involved with this seemingly simple detail: iron worker-structural, iron worker-ornamental, HVAC, and roofing. All need to know what is required and who will be performing what work. First, a roof detail is drawn, as is required for the success of the roof system. All components of this roof detail are noted on the drawing. Next, those notes involving the roofer are made **bold** text with the text of notes pertaining to the other trades remaining neutral. Then the detail is copied but revised to reflect the HVAC contractor's work. On the HVAC contractor's copy, notes referring to HVAC work are printed in **bold** with all remaining notes neutral. The detail is copied a third time, similarly modified for the structural aspects of the project. At this point there are three copies of the same detail, each identifying the scope of work of the respective trade. (See Figure 2a,b,c) This procedure is performed for each identified detail. Information notes for various consultants are also included.

Figures 2a – c: Utilizing coordinating details across the construction drawings informs all trades of their responsibilities.

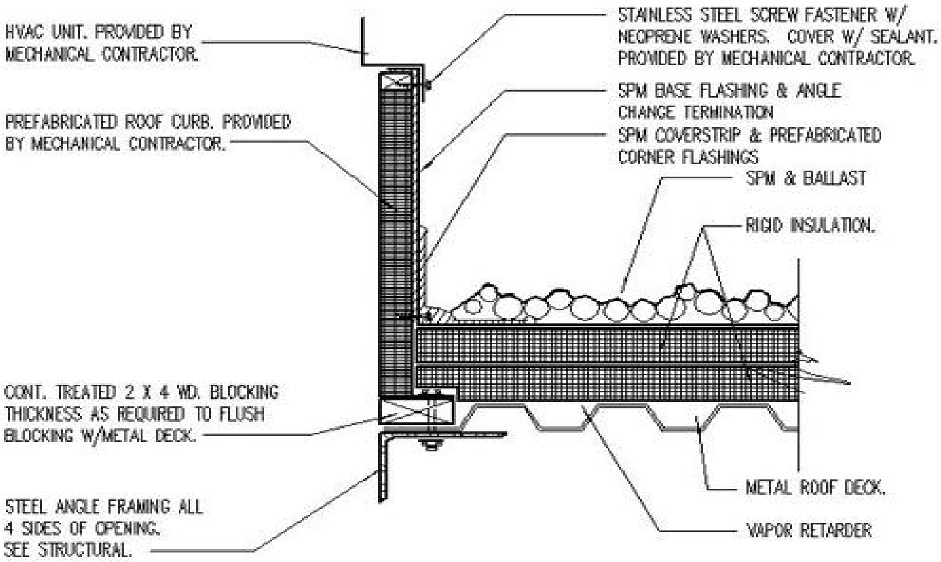


FIGURE 2A. ROOF CURB DETAIL

ROOFING SCOPE OF WORK – FOR ARCHITECTURAL DRAWINGS

NOTE: HVAC CONSULTANT TO SPECIFY ROOF CURB WITH 16" HEIGHT AND EXTERIOR LINER.

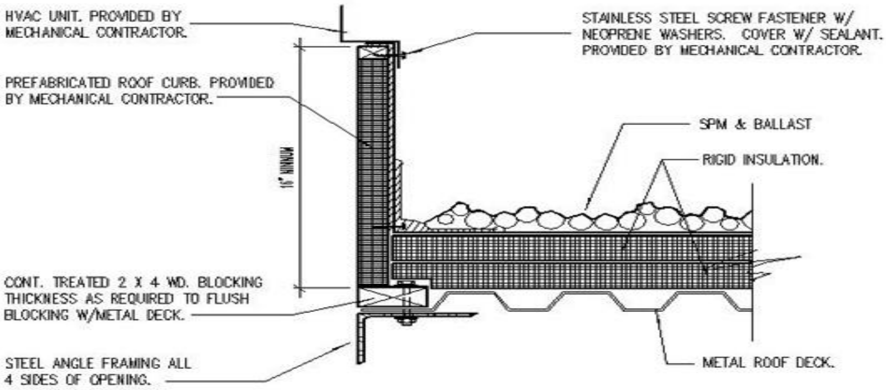


FIGURE 2B. ROOF CURB DETAIL

HVAC SCOPE OF WORK – FOR HVAC DRAWINGS

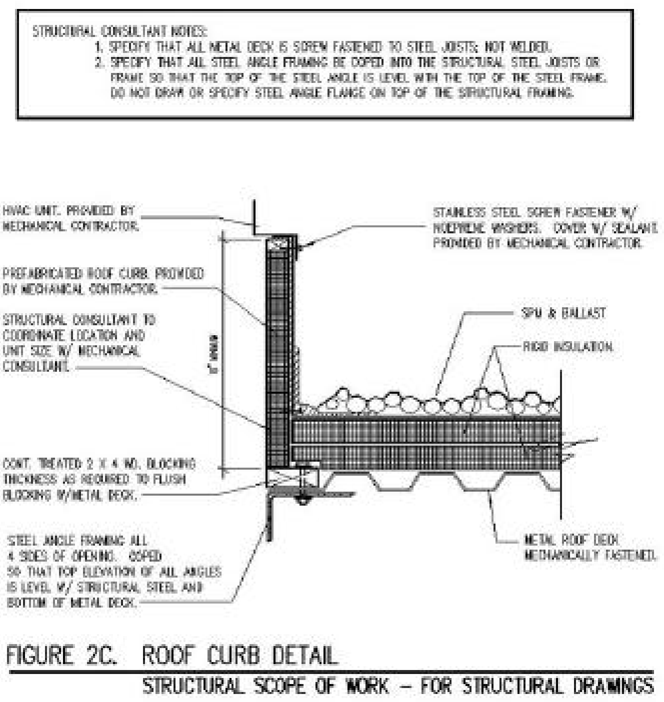


FIGURE 2C. ROOF CURB DETAIL
STRUCTURAL SCOPE OF WORK – FOR STRUCTURAL DRAWINGS

When a detail that impinges on the roof system is identified and checked off on the list, the respective detail is sent to the respective consultants for inclusion on their construction drawings. Coordinating specifications are then written around the details. The impinging component is thus coordinated over the various trades and consultants. Experience shows this system to be extremely useful, time efficient, and successful. The concept has worked for details as common as roof drains and as complex as large rooftop units and their related plumbing. Not all detailing can be anticipated; there are always conditions that will require continuous hard legwork, coordination, and persistence.

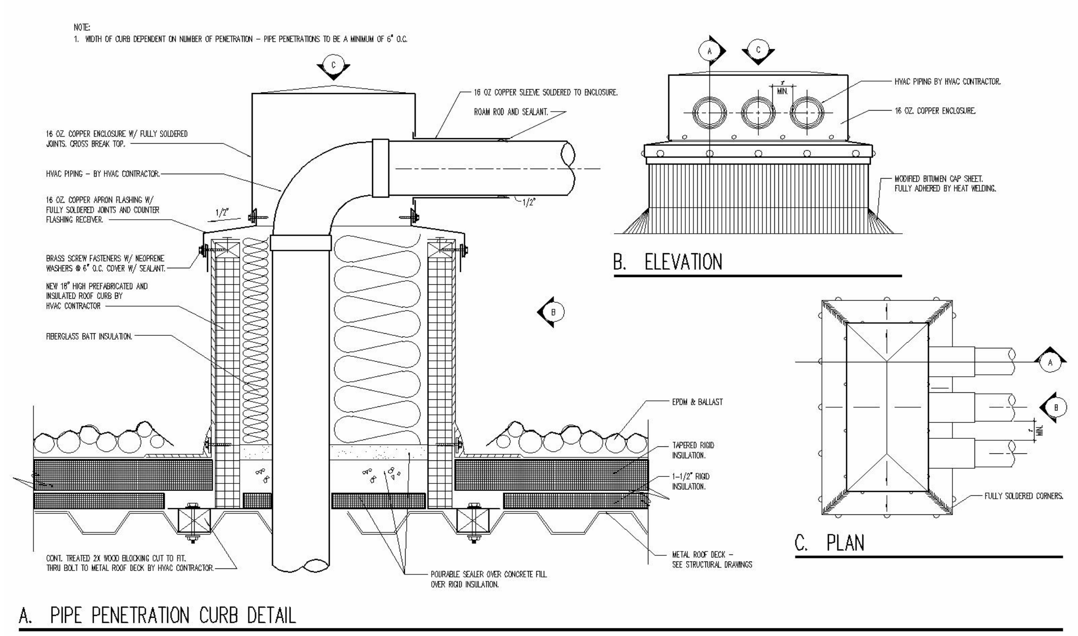


**Figure 3: Anticipating future roofing replacement can save considerable future costs. Thus the installation of a counterflashing receiver prevents the need for removal and replacement of the roof top unit.**

No matter how well roof systems are designed, all have a limited service life. It is recommended that consideration for future reroofing be given during the design/construction phase. For example, large rooftop units are extremely expensive to remove from the roof and reinstall following reroofing. By anticipating possible future needs, counterflashing receivers are installed during the initial construction to accommodate future roofing terminations. (See Figure 3)

While the previous paragraphs have dealt with associated building system components, the building material components need to be considered too. Masonry walls that rise above a lower roof are a case in point. These always pose a challenge to roof system designers. In the construction process the masonry wall is most often constructed before roof system installation. The mason generally does not review roof plans to ascertain whether or not the roof will have tapered insulation. He guesses as to the location of the through-wall flashing height. This can be problematic when the insulation height rises above or near to the height of the through-wall flashing. To prevent these conditions from manifesting themselves as points of moisture intrusion, details should define height of through-wall flashing in measurable terms. Indicating it to be eight bricks above the roof is not acceptable since roof heights vary, roof decks change, and general construction tolerances often render the graphic detail obsolete. Field observations on numerous projects have revealed that the incorporation of a counterflashing receiver with the through-wall flashing results in the termination of the roofing at a proper location. (See Figure 4)

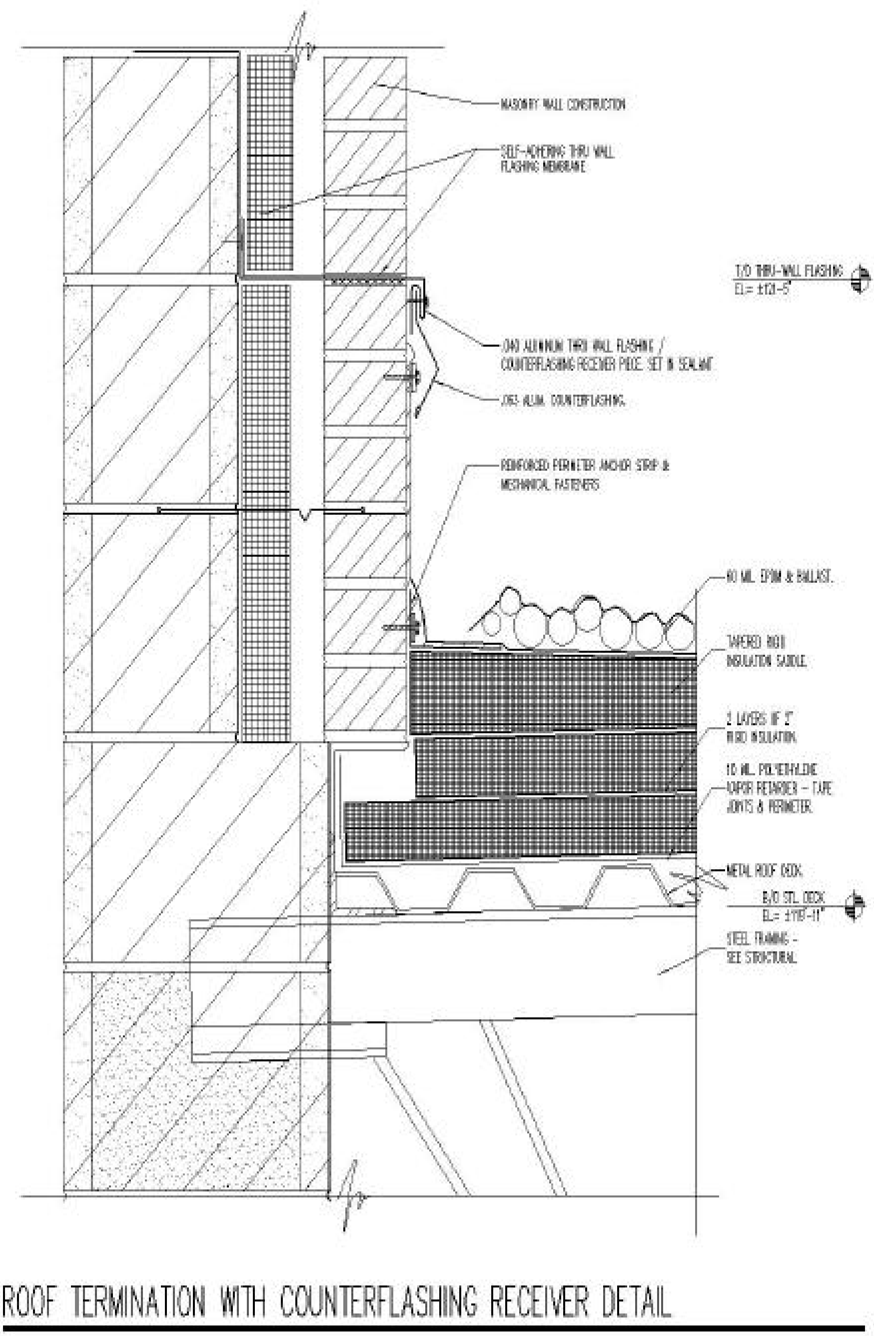


**Figure 4: Incorporating a counterflashing receiver in the through-wall system, noting its height, assists in providing the appropriate fit for roof terminations and provides for future reroofing.**

The concept of acknowledging impinging building components and addressing them in the design-development phase and developing constructible details can be utilized in detailing all types of conditions. The author is currently detailing a roof system termination at a building façade wall in which the building's designer's intent is to have a translucent glass parapet wall. (See Figure 5)

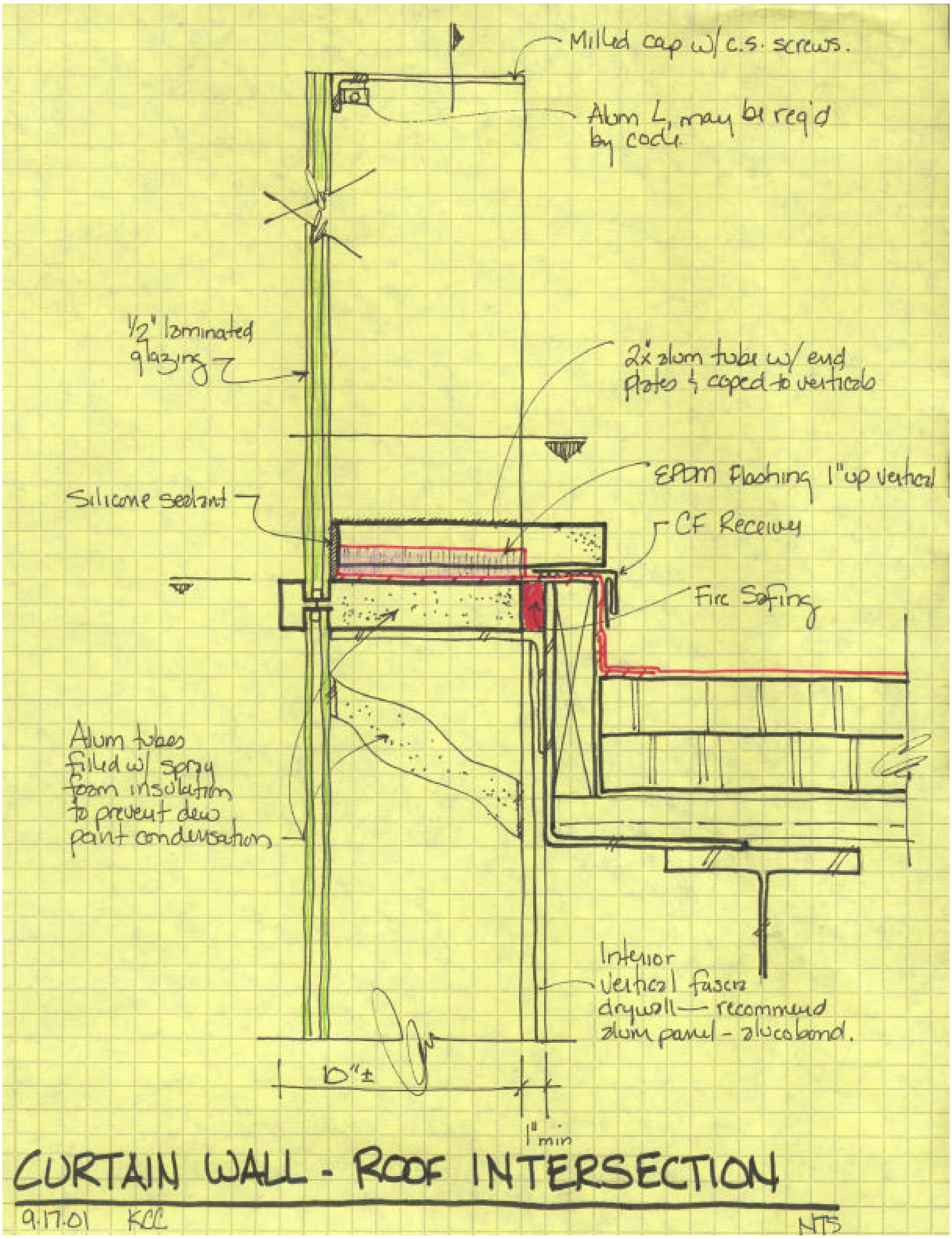


**Figure 5: Addressing how building systems impinge on the roof system early in the building's design and sketching them out to verify their constructability prior to finalization of construction drawings improves the probability of success.**

## 6 CONCLUSION

There is no denying the fact that moisture intrusion through the roof continues to be one of the most frustrating conditions for building users and a litigious one for architects. In most cases the infiltration manifests itself at interface conditions between roof coverings and building components. A systematic approach to identifying these components, detailing them appropriately, and coordinating them during contract document development is the first step in eliminating potential moisture intrusion points of entry.

## 7 RECOMMENDATIONS

The project architect with overall responsibility for the project, client contact, coordination of building component consultant, and scheduling should give consideration to the following recommendations:

A. Roof system designers should get involved in the building design process during the design/development stage of a building's design.
B. Building designers and systems consultants (HVAC, structural, plumbing, etc.) need to be educated to the importance of coordinated detailing.
C. Identify all building components that will impinge on the roof system as soon as possible. Coordinate and resolve conflicts affecting the roof system design.
D. Investigate physical characteristics of the impinging building components so that appropriate roof covering interfaces can be designed.
E. Continuously build relationships with systems consultants to facilitate coordination.
F. Develop a method of standardized detailing and building component coordination such as that described by the author. Modify as needed to be project specific.
G. Do not forget impinging building material components and detail for constructible success.
H. Coordinate all specifications involved.
I. Perform a final review of drawings and specifications, coordinating responsibilities across all disciplines.
J. Perform a continuous self-review for means and methods of improvements.

While a project architect knowledgeable in roofing systems and moisture protection may himself undertake the recommendations, the author has found it most beneficial to have the roof system designer do so, coordinating his work with the design team, project architect, and systems consultants.

# Predicting Current Serviceability And Residual Service Life Of Plywood Roof Sheathing Using Kinetics-Based Models

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**Summary: Research programs throughout North America are increasingly focusing on understanding and defining the salient issues of wood durability and maintaining and extending the serviceability of existing wood structures. This report presents the findings and implications of a 10-year research program, carried out at the USDA Forest Service, Forest Products Laboratory, to develop kinetics-based service-life models for untreated and fire-retardant- (FR) treated plywood roof sheathing exposed to elevated in-service temperatures. This program was initiated because some FR-treated sheathing products were experiencing significant thermal degrade and needed to be replaced. This 10-year research program systematically identified the cause of the degradation and has resulted in new acceptance and performance standards and revisions to U.S. building codes. The strength loss was cumulatively related to FR chemistry, thermal exposure during pretreatment, treatment, and post-treatment processing, and in-service exposure. Quantitatively, a kinetics-based approach could be used to predict strength loss of plywood based on its time–temperature exposure history. The research program then developed models to assess current condition, predict future hazard based on past service life, and predict residual serviceability of untreated and FR-treated plywood used as structural roof sheathing. Findings for each of these subjects are briefly described in this report. Results of research programs like this one can be used to extend the service life of wood by providing engineers with an estimate of residual serviceability and thereby avoiding premature removal. Many of the approaches in these kinetics-based service-life models for plywood roof sheathing are directly applicable to the development of predictive durability models for wood and wood composite roof and wall sheathing that has been exposed to moisture and has eventually decayed. When those models are developed, they will help building code officials, regulators, contractors, and engineers in determining replacement schedules for wood undergoing biological attack.**

**Keywords: Service life, serviceability, durability, fire retardant, treatment**

## 1 INTRODUCTION

North American building codes often require FR-treated plywood roof sheathing for 1.2 m on either side of fire-rated common property walls in multifamily dwellings. Some commercial FR treatments have failed in this use due to premature thermal degradation in as little as 2 to 8 years (Fig. 1). Serviceability assessment methods were needed to evaluate the condition of FR-treated plywood currently in use and to estimate its residual service life. An intensive 10-year research program (Winandy 2001) was undertaken to develop models to

- assess the current condition of FR-treated plywood roof sheathing,
- relate strength loss to treatment, duration of exposure, and exposure temperature and humidity, and
- predict current serviceability and residual service life.

Five critical needs were identified for assessing current condition and developing a predictive residual service-life model for FR-treated plywood roof sheathing (Winandy 2001):

define the mechanisms of thermal degradation,

define the relative importance of treatment, chemical, and processing factors,
develop methods and models for condition assessment,
define service and design factors and thermal loads, and
develop models for predicting residual serviceability.

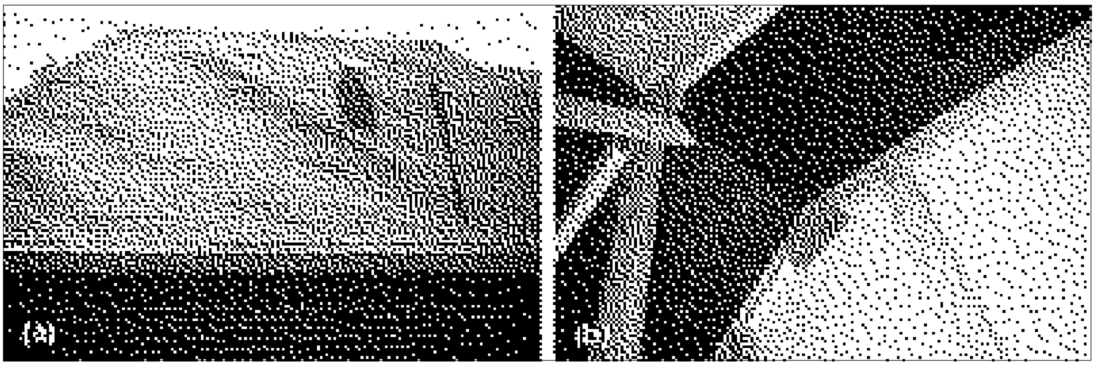


**Figure 1. a) outside view of failed FR-treated plywood roof sheathing; and b) top left—inside view of fully serviceable untreated plywood roof sheathing; center—thermally degraded FR-treated plywood roof sheathing; and lower right—gypsum-sheathed 60-minute-rated fire wall.**

## 2 MECHANISMS OF THERMAL DEGRADATION

Effects of FR treatments on strength properties depend on FR chemistry and thermal processing. FPL research confirmed that field problems with FR-treated plywood roof sheathing resulted from thermal-induced acid degradation.

More than 6,000 specimens of density-matched southern pine (16 by 35 by 250 mm) were systematically exposed at one of four temperatures: 27°C, 54°C, 66°C, and 82°C for exposures up to 4 years (LeVan *et al.* 1990, Winandy 1995, Lebow & Winandy 1999). Data on the rates and magnitudes of thermal-induced strength loss at the four temperatures were obtained for specimens treated with one of six FR model chemicals and untreated specimens. The influence of temperature and treatment pH was progressive, as shown in Figure 2 in which each treatment, at 66°C, has a progressively higher pH (going from left to right on *z* axis). The influence of humidity at temperatures in the 50°C to 80°C range was found to be much less critical to modeling the rate of thermal degrade than the contribution of temperature (Winandy *et al.* 1991).

Kinetics-based models for predicting strength loss as a function of exposure temperature and duration of exposure were then developed from these data obtained at four temperatures. These kinetic models can be used to predict thermal degradation at other temperatures (Lebow and Winandy 1999). A single-stage approach quantitatively based on time–temperature superposition was used to model reaction rates:

$$Y_{ij} = b_j * \exp(-X * A_j * (H_t / H_o) * e^{(-Eaj/RTij)}) \qquad (1)$$

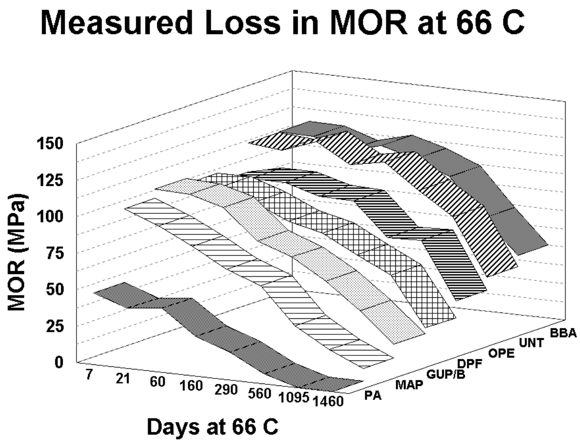


**Figure 2. Experimentally measured bending strength loss with time of exposure to 66°C (Winandy 1995, Lebow & Winandy 1999) (PA phosphoric acid, MAP monoammonium phosphate, GUP/B guanylurea phosphate/boric acid, DPF dicyandiamide-PA-formaldehyde, OPE organophosphonate ester, UNT untreated, BBA borax/boric acid).**

where

| | |
|---|---|
| i | = temperature of exposure, |
| j | = FR chemical, |
| $Y_{ij}$ | = bending strength (MPa) at temperature ($T_i$) for $FR_j$, |
| $X$ | = time (days) at temperature ($T_i$) for $FR_j$, |
| $b_j$ | = initial bending strength (MPa) at time ($X_i = 0$), |
| $H_t$ | = relative humidity at test, |
| $H_o$ | = normalized relative humidity (67% RH per ASTM D5516), |
| $A_j$ | = pre-exponential factor, |
| $E_{aj}$ | = activation energy for $FR_j$, |
| $R$ | = gas constant (J/K*mole), and |
| $T_{ij}$ | = temperature (K) and for $FR_j$. |

This kinetics-based model appeared to fit the combined data set (54°C, 66°C, and 82°C) as well or slightly better than alternative approaches (Lebow & Winandy 1999). However, this model should not yet be extrapolated beyond its intended temperature range (40°C–90°C) because it appears to overpredict strength loss at temperatures below 40°C (Winandy & Lebow, unpublished data).

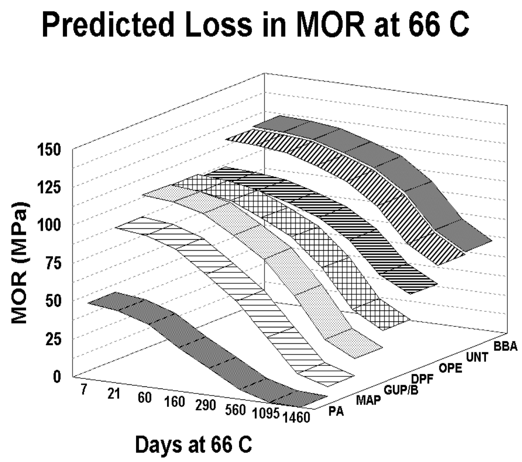


**Figure 3. Predicted bending strength loss with time of exposure to 66°C (Lebow & Winandy 1999). (PA phosphoric acid, MAP monoammonium phosphate, GUP/B guanylurea phosphate/boric acid, DPF dicyandiamide-PA-formaldehyde, OPE organophosphonate ester, UNT untreated, BBA borax/boric acid).**

With help from our academic and industrial cooperators, this program has resulted in three new ASTM Standard Test Methods: D5516 for evaluating FR-treated plywood, D5664 for evaluating FR-treated lumber, and D6305 for deriving engineering design adjustments for FR-treated plywood (ASTM 2001). Also, all currently accepted AWPA FR formulations have now been evaluated under these methods (AWPA 2001).

## 3 PROCESSING FACTORS

Our research has proved that the use of pH buffers in FR chemicals, such as borates, can partially mitigate the initial effect of the FR treatment on strength and then significantly enhance resistance to subsequent thermal degradation. For modulus of elasticity, there appeared to be few real benefits derived from adding borate-based pH buffers to the FR mixture on the subsequent thermal degrade of FR-treated plywood exposed to high temperatures (Winandy 1997). However, after 290 days of exposure at 66°C, there were significant ($P < 0.05$) benefits derived with respect to limiting strength loss and loss in energy-related properties, such as work to maximum load, by the addition of borate to the FR–chemical mixture. Remedial treatments based on surface application of pH-buffered borate–glycol solutions were also developed to protect against additional in-service strength loss (Winandy & Schmidt 1995).

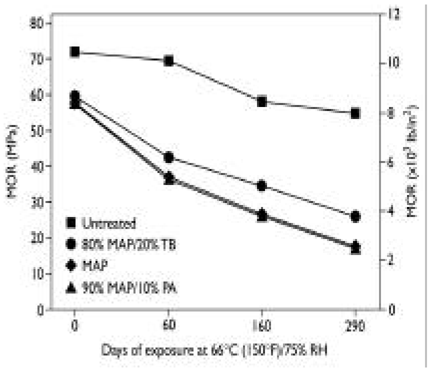


**Figure 4. Effect of pH buffers on rate of thermal degrade (Winandy 1997) (MAP, monoammonium phosphate; PA, phosphoric acid; TB, sodium tetraoctaborate).**

Other work also found that the rate of strength loss was largely independent of plywood quality or grade (Lebow & Winandy 1998). Also, variation in redrying temperatures from 49°C to 88°C had little differential effect on the subsequent rate of thermal degradation when the treated plywood was exposed at 66°C for up to 290 days. This was related to the shorter kiln residence times required at higher temperatures which yielded similar states of entropy via differing, but thermodynamically comparable, temperature–duration histories.

## 4 CONDITION ASSESSMENT

Before future strength loss could be predicted, current condition had to be assessed. We found that screw withdrawal tests (Fig. 5) were reliable indicators of degradation (Winandy *et al.* 1998).

This study defined constitutive relationships between nondestructively measured properties and the bending strength of FR-treated plywood (Fig. 6). These constitutive relationships between screw withdrawal force and residual bending strength were then used in a manner similar to how modulus of elasticity is used to predict bending strength in machine-stress-rated lumber grading. The final step toward implementation will be for researchers and the engineering community to work together to develop consensus precision estimates to enable third party interpretation of these constitutive relationships.

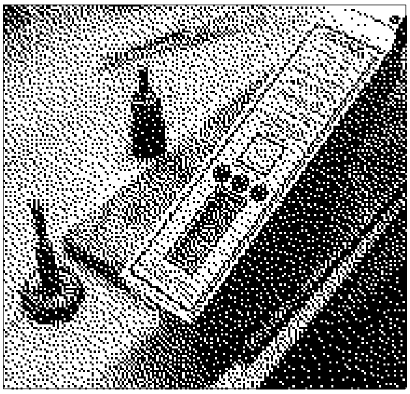

**Figure 5. Screw-pull test using hand-held load cell.**

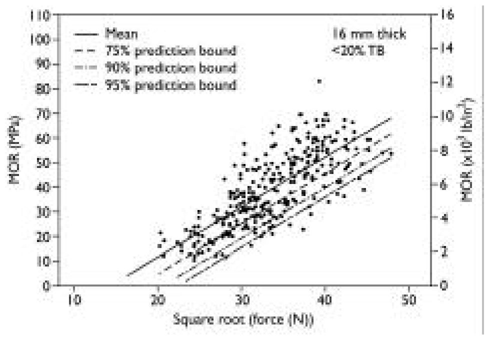


**Figure 6. Constitutive relationships for screw-pull tests (Winandy *et al.* 1998).**

## 5 SERVICEABILITY FACTORS AND THERMAL LOADS

Our first goal was to define relationships between field and laboratory exposures. Special roof temperature monitoring chambers were constructed in Madison, Wisconsin (latitude 43.4° North) and Starkville, Mississippi (latitude 33.5° North) (Winandy & Beaumont 1995, Winandy *et al.* 2000). These chambers monitored temperatures for the structural plywood and wood rafters used in traditional North American wood-framed construction under asphalt–fiberglass shingles (Fig. 7).

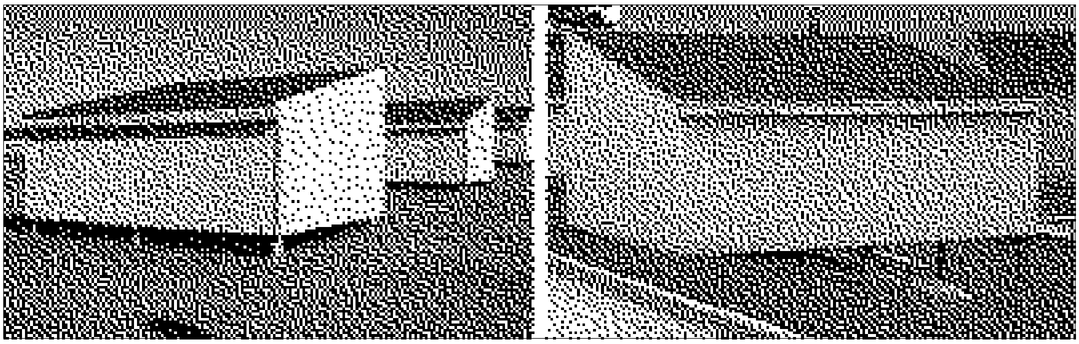

**Figure 7. Field chambers for monitoring roof temperatures in Wisconsin on left (northern U.S., 43.4°N) and Mississippi on right (southern U.S., 33.5°N).**

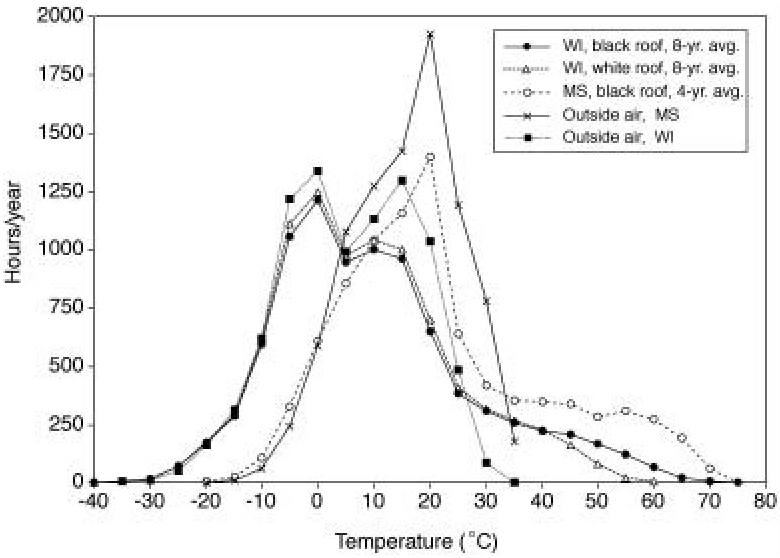


**Figure 8. Annualized roof temperature data for plywood roof sheathing in Madison, Wisconsin (Winandy *et al.* 2000)**

Roof temperature data are now available for 8 years in Madison, Wisconsin and 4 years in Starkville, Mississippi (Fig. 8) (Winandy *et al.* 2000). The maximum temperatures recorded in our 4-year Mississippi study for black-shingled roofs in dry buildings were 78°C, 63°C, and 58°C for the top-ply veneer, bottom-ply veneer, and inside nominal 2- by 8-in. (standard 38- by 184-mm) rafters, respectively. The maximum temperatures recorded for the matched Wisconsin roof systems during the 8 years were 75°C, 59°C, and 54°C, respectively. The maximum temperatures recorded in our 4-year Mississippi study for black-shingled roofs in heavily humidified buildings were cooler than in matched dry buildings at 74°C, 58°C, and 54°C for the top-ply veneer, bottom-ply veneer, and internal 2 by 8 rafter temperatures, respectively. Differences in daily maximums and annualized temperature data for each wood component were similar to those of the previously reported 3-year Madison data (Winandy & Beaumont 1995). These results clearly indicated that the temperatures of wood components used in wood roof systems were more influenced by the influx of radiant solar energy than by ambient outside air temperatures (Winandy *et al.* 2000).

The second goal was to verify and refine the FPL temperature history model to predict in-service temperatures of wood roof system components. The FPL Roof Temperature Model predicts roof temperatures for plywood roof sheathing based on geographical factors, site factors, orientation to sun, building construction, and historical weather data (TenWolde 1997). That model has now been published and is used as the tool for predicting temperature histories for roof sheathing in untested locations and for designs in our new residual serviceability models.

# 6 PREDICTING RESIDUAL SERVICEABILITY

The goal of this final project is to develop the best model to evaluate residual service life of FR-treated roof sheathing plywood. A residual serviceability prediction model was recently developed to predict on-going strength loss from solar-induced thermal loads and to compare candidate FR systems (Fig. 9). We used our models (Eq. 1) to simulate up to a 20-year exposure using the 8-year annualized data for both Wisconsin and Mississippi (Fig. 8). Our residual serviceability model predicted that plywood treated with 56 kg/m$^3$ phosphoric acid (PA) could be expected to experience an 11.3% loss from its original in-service load capacity after the 20-year simulation under a black roof in Wisconsin. Predicted strength loss for untreated wood was only 10.6% after the 20-year simulation under a black roof in Wisconsin. In Mississippi, which has many more warm days per year than does Wisconsin, those losses after 20 years were 31.8% for PA and 28.2% for untreated plywood under a black roof. Plywood treated with other FR chemicals, such as 56 kg/m$^3$ of monoammonium phosphate (MAP) or a 70/30 mixture of guanylurea phosphate/boric acid (GUP/B), experienced intermediate levels of strength loss. Information of this type is currently being discussed in U.S. design codes and standards.

**a. Wisconsin (43.4°N** **b. Mississippi (33.5°N)**

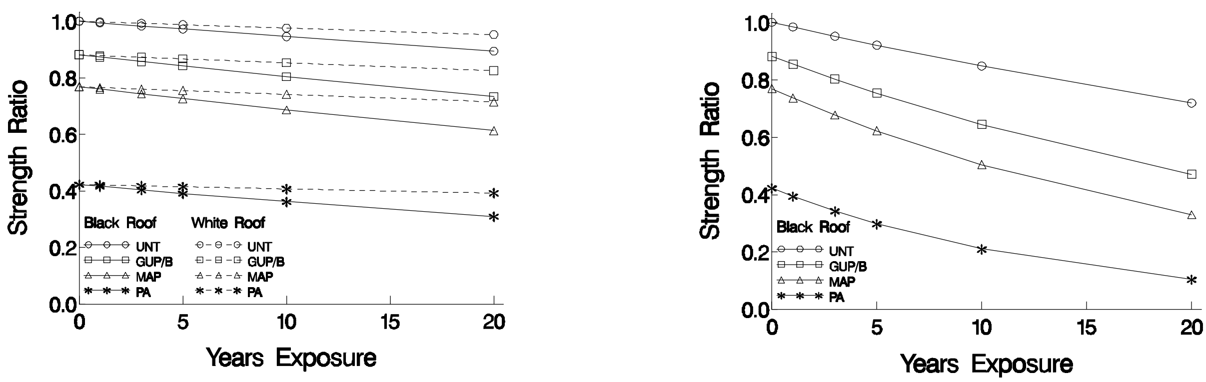


**Figure 9. Predicted plywood strength after 20 years of in-service exposure using a preliminary residual serviceability prediction model. Each treatment has progressively lower pH (in the order of GUP/B, MAP, PA) (J.F. Murphy, P.K. Lebow, J.E. Winandy, Development of residual service-life models for plywood roof sheathing (unpublished)) (UNT untreated, GUP/B guanylurea phosphate/boric acid, MAP monoammonium phosphate, PA phosphoric acid).**

The predicted strength losses and projected loss in field serviceability obtained by applying the kinetic degrade models discussed in the section Mechanisms of Thermal Degrade to measured roof temperature histories paralleled actual field performance. An extensive model development project is now underway to more fully define and refine the residual serviceability model for FR-treated roof sheathing exposed to elevated in-service temperatures.

# 7 SUMMARY

A kinetics-based service-life model can provide a valuable tool for building code officials, regulators, contractors, and engineers in determining replacement time schedules for wood and wood composites undergoing acid-catalyzed thermal degradation. Many of the concepts employed in the development of residual serviceability models for plywood roof sheathing are conceptually applicable to the development of predictive durability models for wood as affected by moisture and eventually by decay.

# 8 ACKNOWLEDGMENTS

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# A Prediction Model For Life Cycle Costs Based On Design Quality

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**Summary**: **Operating, maintenance and rehabilitation (O,M&R) costs can account for up to 85 percent of total life cycle costs for a facility. It is a commonly held tenet that the majority of these costs are pre-determined at the design stage and, that the ability to influence them declines dramatically as a facility moves through the design, construction and commissioning stages to the eventual O,M&R of the building. So many factors contribute to overall building quality that proving this relationship is difficult. Three key obstacles to establishing a relationship between quality and life cycle costs are: (1) the difficulty in finding a large enough sample of common building designs to facilitate appropriate comparisons between buildings; (2) the difficulty in finding common construction and O,M&R policies and procedures; and (3) the lack of historical life cycle cost data.**

**These obstacles were overcome by partnering with the Canadian Department of National Defence to study buildings on selected military bases in Eastern Canada. In total, 215 buildings, ranging from two to fifty years in age formed the database for the research. Construction, operating, maintenance, rehabilitation and demolition costs were gathered for all of the facilities and total costs were calculated. Formulae were developed to quantify the degree of quality in the initial building design, construction and O&M based on a review of as-built drawings, historical contract data and a survey of users and managers. The formulae yielded a quality score referred to as the "quality quotient" or QQ which could then be plotted against life cycle costs to determine if a relationship between the two variables could be modeled.**

**This paper presents the preliminary results for 15 administration buildings. The preliminary findings indicated that a relationship between life cycle costs and quality was difficult to establish based on the limited data set and variables. It was found that building function had a greater impact than anticipated on O,M&R costs and that more accurate operating costs needed to be determined before life cycle costs could be calculated. The research is continuing so that the data set can be expanded and the model refined to include functionality.**

**Keywords. asset management, service life, design quality, life cycle cost modeling, quality quotient**

## 1 INTRODUCTION

In recent years, the Canadian government has succeeded in eliminating Canada's fiscal deficit; however, it has yet to seriously tackle what many engineers refer to as the "infrastructure deficit". Most of the nation's public infrastructure is approaching the end of its life cycle and warrants major rehabilitation or replacement. The Canadian Department of National Defence (DND), the largest owner of federal infrastructure, is coping with major reductions in funding at a time when its infrastructure is aging, as shown in Fig 1, and in need of rehabilitation. While increased funding would address the problem, how current and new allocations are spent must also be examined. Life cycle cost analysis; service life determination; and established maintenance and rehabilitation (M&R) policies and procedures are effective tools for asset managers to use in determining the most cost-effective way to maintain infrastructure assets.

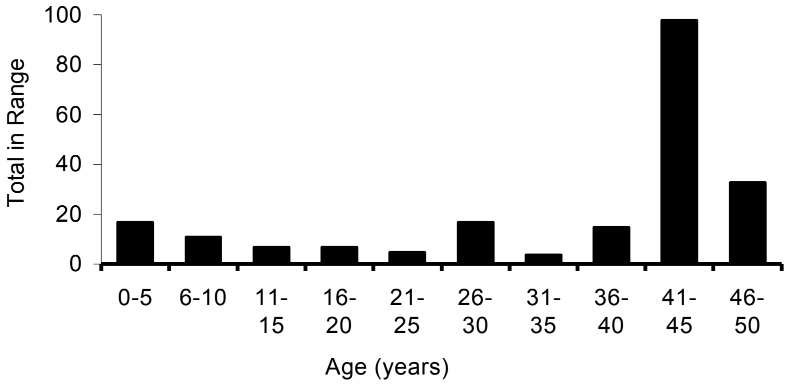


**Figure 1 Distribution of DND study sample by age of building**

Life cycle costs (LCC) typically include all expenditures, from preliminary design through to the construction, operation and maintenance (O&M), rehabilitation and disposal or demolition of a facility. The ability to influence life cycle costs is greatest during the design phase as the types of materials specified, the quality of the design and the contracting method chosen impact directly upon future O&M costs. Within DND, the seemingly indestructible concrete barrack block design of the 1950's has been replaced with a timber-framed structure, for which long term maintenance costs may well exceed the savings in initial construction costs due to the use of less durable construction materials, operating costs notwithstanding. The change from the traditional approach to design and construction, wherein a contract is awarded to the lowest bidder, to partnering and design-build contracts also has life cycle cost ramifications.

Facilities in the study were not commercial properties and therefore, the consideration of the cost of land and interest on loans were not necessary. The quest for quality to reduce life cycle costs might be more rewarding for DND, who will own facilities for life, compared to commercial enterprises/properties/facilities where ownership is likely to change hands throughout the life of the facility. Furthermore, decisions taken in the private sector as to whether to undertake rehabilitation or renewal of infrastructure are influenced more by economic factors such as property values than those in a public sector organisation such as DND.

It is readily accepted that considerations such as the quality of design, materials used and workmanship will have a significant impact upon future operation and maintenance (O&M) costs and ultimately, the life cycle cost of whatever is being purchased. Asset managers are forced to make cost reduction decisions in design and construction knowing full well that the choices they make today have the potential to impact significantly upon future O&M costs and thus, life cycle costs. Facility and asset managers must also determine how and when to repair older infrastructure while striving to minimise costs and maximise value with scarce O&M funding allocations. Furthermore, they have to balance the need to maintain older infrastructure with requirements for new construction. Thus, a simple formula that uses a building design quality score to predict life cycle costs has the potential to be a valuable tool for asset managers (Christian & Newton, 1999).

Previous research within the Construction Engineering and Management Group at the University of New Brunswick has found that the greatest impediment to research on the relationship between life cycle costs and quality is the lack of accurate historical cost data and commonality in the approach to O&M (Christian et al, 1998). DND however, has maintained reasonably accurate historical cost records of all of its facilities on bases located throughout Canada. The data include property information, design and construction details, construction costs, maintenance costs and operating costs. For some bases, the data extend as far back as the 1950's. In addition, DND has well established maintenance policies and procedures that promote consistency in the O&M of its facilities. It has only been in the past five years however, that DND had begun to critically examine its procedures to determine whether or not they lead to mininizing life cycle costs.

A recent downsizing of the Canadian Forces (CF) included the closing of many smaller military bases across Canada. The closures precipitated a flurry of new construction to accommodate the relocation of established units to larger bases. These events, combined with changes in design, contract award procedures and construction methods, have provided a once in a lifetime opportunity to study quality, and its impact on the life cycle costs of common facilities, over three stages in the life cycle: newly constructed, mid-life (10-20 years) and end-life (35+ years).

## 2 RESEARCH METHODOLODY

Preliminary LCC research was undertaken at a military base in New Brunswick in the Fall of 1997. From an initial examination of residential facilities, it was determined that sufficient cost and service life data were available to expand the research to other bases. The research parameters were set out in several meetings with the Directorate of Realty Asset Plans. It was agreed that time constraints limited the study to bases in Eastern Canada. Initially nine bases were selected; however, one base was subsequently dropped from the research programme due to the age of its infrastructure and the lack of commonality with other bases.

Five major categories of facilities were selected for study: administration, institutional, residential, maintenance and operations facilities. Multi-use buildings, which combined two or more of the aforementioned functions, were also included. Individual buildings were selected based on the following criteria:

- a common design with at least one other building either on the base or at another base in the study group
- constructed after 1950
- within one of the five research categories or a combination of two or more categories (multi-use)

### 2.1 Data collection

Site visits were arranged to each base to elicit the following data:

- all costs associated with each facility including construction costs, maintenance costs, rehabilitation costs, re-capitalisation costs and demolition costs (if applicable) up to fiscal year 1998/1999
- operating costs including heating, water, sewage, electricity and janitorial costs up to fiscal year 1998/1999
- usage figures for residential and administration buildings
- building design details
- photographs

The data were obtained from old property record cards, the current asset management database (Aladdin), development files, old project files, original contracts on file, preventive maintenance (PM) records, historical cost data from the construction engineering management information system (CEMIS) and base construction engineering management system (BCEMS), in addition to current cost data and "as-built" drawings. The visits took place during the period December 1998 – September 2000. From an initial sample of over 400 buildings, data were collected for 215 buildings. Out of this number, sufficient historical data was available to facilitate life cycle cost analysis, service life determination and an analysis of building quality for approximately 175 buildings.

### 2.2 Building design, construction and O&M analysis

A building design analysis was undertaken for each building using the "as-built" drawings obtained during the site visits. The Canadian Standards Association (CSA, 1995) and the British Standards Institution (BSI, 1992) standards were used as guidelines for the analysis and included the following design conditions and characteristics: category of building; building design service life (DSL) and factors affecting building DSL; and critical component construction, failure mode, DSL and required maintenance level. The data obtained from the analysis were used to assess the design quality of the building.

Due to the age of the building inventory, the primary source of data for construction was historical contract documentation and subjective evidence from contract and engineering management personnel. Where available, the original contract documents, change orders and design drawings were reviewed. The selection of the critical building components was based on the impact that failure would have on the operation of the facility. They were determined to be the foundation and slab; the supporting structure; exterior cladding; the roof; windows and doors; heating, cooling and ventilation systems, overhead and aircraft hangar doors; and electrical systems.

The DND has well-established O&M policies and procedures which emphasize preventive maintenance. Thus, many buildings had the appearance of being well maintained. An analysis of O&M records was undertaken to determine whether or not the policies and procedures were applied consistently from base to base.

### 2.3 Cost analysis

For life cycle costing purposes, DND assigns a 40-year DSL to all of its buildings with a mid-life retrofit considered at the 20-year point. In reality, the majority of DND buildings exceed this DSL. If it is determined that the building can still meet its function or can be renovated to accommodate another function, a major retrofit is undertaken to extend the service life of the building. Where two or more functions (e.g. administration and operations) can be consolidated, new facilities are proposed to replace the older buildings.

Building construction, maintenance and rehabilitation (C,M&R) costs were determined from actual expenditures recorded over the service life of each building. All historical costs were converted from the year in which they were incurred into constant dollars (1999CA$) using the DND Economic Model. Costs were then converted into annual equivalent costs (AEC) using the DSL period of 40 years and a 25-year discount rate, obtained from DND, of 5.6%.

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The formulae used to calculate AEC for C,M&R were as follows:

$$AEC(\text{Const}) = ICC * EF_N * crf \qquad (1)$$

$$AEC(\text{Maint or Rehab}) = crf * \{ \mathop{?}\limits_{N=1}^{40} (M \text{ or } R)_N * EF_N + [ICC*p*(40-ASL)]\} \qquad (2)$$

where:

- ICC is the initial construction cost of the building
- $EF_N$ is the appropriate DND economic factor for the year of construction, maintenance or rehabilitation
- *crf* is the capital recovery factor $\frac{i(1+i)^N}{[(1+i)^N - 1]}$ for a period N= 40 years and i = 5.6%
- $(M \text{ or } R)_N$ is the historical maintenance or rehabilitation cost incurred annually from the year of construction to the end of the DSL
- p is a percentage equal to 1% of the ICC for a new building or the average M&R spent on the building to date as a percentage of the ICC. It is used to approximate the present value of M&R expenditures up to the 40-year DSL
- ASL is the actual service life of the building

Thus, the annual equivalent cost calculation for a hypothetical building constructed in 1960 at a cost of $\$100/m^2$ is obtained as follows:

E.g. $$AEC(\text{Const}) = \$100 * 7.788 * \frac{0.056(1.056)^{40}}{(1.056)^{40} - 1} = 1999CA\$48.36/m^2$$

Annual equivalent maintenance (or rehabilitation) costs of $5.00 and $10.00 incurred at year 5 and 20 of the 40 year DSL respectively would be calculated thus:

E.g. $$\begin{aligned} AEC(\text{Maint}) &= crf * \{(5.00 * 7.045) + (10.00 * 2.451) + [\$100 * 0.01 * (40 - 40)]\} \\ &= 0.0631 * (35.23 + 24.51 + 0) = 1999CA\$3.77/m^2 \end{aligned}$$

Since historical operating costs for individual buildings were not available, annual equivalent operating costs were obtained from the DND Cost Factors Manual (CFM). The CFM only provides costs by location and building category. In addition, the format of the CFM has changed constantly over the past decade. This presents a problem as the following energy-related factors are not taken into account in the CFM:

- the impact of rehabilitation to improve energy performance in older buildings
- the energy efficiency of newer buildings
- the impact of a central heating plant as a source of heat
- the increase in use in electronic equipment and climate control systems
- the historic cost of energy

In an attempt to better approximate the impact of energy performance on annual operating costs, heat loss calculations were undertaken for a standard building, an energy-retrofitted building and a newly constructed administration building. To assess the impact of energy demands, detailed load inventories and historical energy costs were studied. A review of the U.S. Department of Energy figures, for the period 1946-1999, found that the fossil fuel composite price in constant 1996$US had remained relatively stable, with the exception of the abnormally high energy costs of the mid-1970's to mid-1980's. Over the same period however, fossil fuel demand, had increased tenfold (U.S.DoE, 2000).

The historical costs were compiled and entered into a Microsoft Access® database by building, cost type (construction, maintenance, operations, etc) and description (building envelope, interior finishes, superstructure, etc). Building costs were then calculated and converted to annual equivalent costs of $1999\$/m^2/yr$ to facilitate comparison between buildings of similar age and type. Individual design and contract inspection costs were not included in the analysis as they were unavailable for individual buildings. Typically, DND pays consultant design costs in the vicinity of 5 to 10% of the final contract price and Defence Construction Canada (DCC) contract inspection costs of 3 to 5%.

## 3 QUALITY QUOTIENT DEVELOPMENT

Methods to measure building and component quality and service life have been proposed by several researchers, namely Bourke & Davis (1997), Baird, et al (1996) and the British Housing Association Property Mutual (1995). Each method attempts to quantify quality, primarily through a detailed analysis of the building design, during or shortly after the completion of the design stage. It is not always possible, nor practicable, to undertake such an analysis for buildings that are already constructed. In addition, total quality in a facility is an integrated function of quality in design, construction and O&M. It is proposed therefore, that quality can be represented by a simple quality quotient (QQ) shown in the following equation:

$$QQ = xQ_DQ + yQ_CQ + zQ_{OM}Q \tag{3}$$

where:

- *x*, *y* and *z* are constants based on the level of influence of quality has on facility life cycle costs when considered in each of the phases of design, construction and O&M respectively.
- QQ is the quality quotient , with a maximum value of 1.0
- $Q_DQ$ is the design quality quotient, with a maximum value of 1.0
- $Q_CQ$ is the construction quality quotient, with a maximum value of 1.0
- $Q_{OM}Q$ is the operation and maintenance quality quotient, with a maximum value of 1.0

The constants were determined from previous findings (ASCE, 1988) to have values of $x = 0.8$, $y = 0.15$ and $z = 0.05$ respectively. A random survey of construction contractors, designers and facility owners/managers is currently being undertaken to validate the numbers used in the equation.

### 3.1 Quality Dimensions

Acccording to Garvin (1984) and McGeorge & Palmer (1995), quality is comprised of seven dimensions: performance, reliability, conformance, durability, serviceability or maintainability, perceived quality and aesthetics.

#### 3.1.1 Performance

Performance is the primary operating characteristics of a building that enable it to achieve its main purpose and meet the needs of the users. A residential facility may function as a residence; or it may function in a secondary role as an administration building and still achieve its purpose.

#### 3.1.2 Conformance

Conformance is the degree to which a building conforms to the original statement of requirement or specification. This implies that only a few design changes were made to the original design documents.

#### 3.1.3 Reliability

The probability of a building or its components failing within a specified period of time. It is not to be confused with durability but is best described by relating how a specific component, eg. door hinges, has functioned in comparison to the expected rate of failure for similar components.

#### 3.1.4 Durability

Durability is a measure of design service life or the length of time a building, or its components, lasts before it physically deteriorates. Durability is usually specified by the manufacturer and supported through standardized testing. As an example, a roof with a DSL of 20 years should last 20 years before rehabilitation or replacement is needed. The longer a component lasts, the more durable it is said to be.

#### 3.1.5 Serviceability

In facility management, serviceability is frequently called maintainability and is a measure of the speed. and competence of repairs that ensures repairs are made quickly and with a minimum of disruption to the users. It also implies that components that will require repair or replacement, eg. water pipes, should be easily accessible for service personnel.

### 3.1.6 Aesthetics

Aesthetics are a measure of how the building looks, feels and sounds to the users. The priority given to aesthetics depends upon the function of the building. Aesthetics may not be a prime consideration in the design of a garage but would be in an office.

### 3.1.7 Perceived quality

Percieved quality takes into account a user's perception of the building. Design, construction and quality of life are all aspects that a user considers when forming an opinion on the quality of a building.

## 3.2 Determination of quality scores

Determination of the quality scores began with a through analysis of all relevant cost and component data available for a building. This included a study of key factors affecting the building design and the identification of critical components, their design and the levels of maintenance required. The relevance of each quality dimension in the three life cycle phases and whether or not it was measurable was then considered. A metric was develped for all seven dimensions and applied, where measurable, to all three phases of the life cycle. A sample metric for the "conformance" dimension is illustrated at Table 1.

**Table 1. Quality metric for "conformance" dimension**

| *Design quality - $Q_D$* | *Construction quality - $Q_C$* | *O&M quality - $Q_{OM}$* |
|---|---|---|
| Conformance Score – based on a review of the original property record cards, Statement of Requirement (SOR) and a review of contract documentation<br><br>5 - building conforms to SOR, no major design changes during construction<br><br>4 - building conforms to SOR, minor design changes during construction<br><br>3 - building conforms to SOR with minor design changes during construction during first year of operation due to previously unfound errors in design<br><br>2 - building conforms to SOR but major changes made after design finalised or within first two years of operation due to previously unfound errors in design OR building "over designed<br><br>1 – building did not conform to SOR | Conformance Score – based on a review of contract documentation to determine success of project delivery<br><br>5 – cost and/or schedule growth < 5%, excellent workmanship<br><br>4 - cost and/or schedule growth < 5%, average workmanship<br><br>3 - cost and/or schedule growth 5–10%, excellent workmanship<br><br>2 - cost and/or schedule growth 5–10%, average workmanship<br><br>1 - cost and/or schedule growth >10%, average to poor workmanship | Conformance Score – based on a review of preventive maintenance and property record documentation for a Base<br><br>5 - O&M always conforms to established policies, procedures and best practices<br><br>3 - O&M usually conforms to established policies, procedures and best practices<br><br>1 - O&M seldom conforms to established policies, procedures and best practices |

The raw score of each dimension was then multiplied by a weighting factor to yield the individual quality score or QQ. The following equations were used to calculate the $Q_DQ$, $Q_CQ$ and the $Q_{OM}Q$:

$$Q_DQ = Q_D(pP + rR + cC + dD + sS + aA + pqPQ)/5 \tag{4}$$

$$Q_CQ = Q_C(cC)/5 \tag{5}$$

$$Q_{OM}Q = Q_{OM}(pP + rR + cC + dD + sS + aA + pqPQ)/5 \tag{6}$$

where:

- the constants p, r, c, d, s, a, pq are weighting factors
- P, R, C, D, S, A, PQ are the seven quality dimensions

To determine the constants (p, r, c, d, s, a, pq), a questionnaire was sent to designers, contractors, and owners/facility managers asking them to rank, and assign a percentage to the importance of the seven quality dimensions in influencing building life cycle costs. The weighting factors derived from the results of the questionnaire are presented in Table 2. The values for $Q_DQ$, $Q_CQ$, $Q_{OM}Q$ were then inserted into equation (3) to yield a quality quotient or QQ for each building.

**Table 2. Quality quotient weighting factors - importance of quality dimensions.**

| *Design service life* | *Quality dimension* | | | | | | |
|---|---|---|---|---|---|---|---|
| | *p* | *r* | *c* | *d* | *s* | *a* | *pq* |
| 25 – 49 years | 0.22 | 0.18 | 0.14 | 0.16 | 0.15 | 0.07 | 0.08 |

## 4 FINDINGS

As previously discussed, the goal of the research is to study quality and its impact on life cycle costs of military facilities. In order to examine this link, a preliminary evaluation of the QQ formulae using all of the "administration" buildings in the study group was undertaken. This category consisted of a sample of 15 buildings ranging from newly constructed to 45 years in age. Three of the 15 buildings were less than 26 years of age, one was newly constructed and the remaining eleven were over 40 years of age.

### 4.1 Life cycle costs and annual equivalent costs

Annual equivalent building costs in constant 1999CA$ based on the 40-year DSL, are presented at Table 3. Eleven of the buildings shared a common or standard design, as noted in the *Description* column. This is important because, with the exception of potentially different site conditions, it eliminates design as a factor when considering the total costs incurred by the facility. Theoretically, a standard design should yield similar costs for each building. It can be seen from the table that with the exception of two buildings, (Nos. 6 & 12) at 1999CA$67.12/$m^2$ and 1999CA$97.29/$m^2$ respectively, the construction costs for these particular facilities are close in value, with the average cost for all eleven buildings calculated to be 1999CA$78.15/$m^2$. The construction costs for the remaining four buildings vary widely from 1999CA$55.07/$m^2$ to 1999CA$90.46/$m^2$.

There was also considerable fluctuation in the maintenance and rehabilitation costs of the standard-design buildings. It was found that this was directly attributable to the function of the building. Those buildings with a higher "profile" such as a senior headquarters (Nos. 1, 3, 4 & 8) or those which had undergone a change in profile (Nos. 11 & 12) had higher combined maintenance and rehabilitation costs. Buildings that housed operational units, such as an infantry battalion or an artillery regiment, were not fully occupied for up to 50% of the year (Nos. 2, 7, 9 & 10) and thus, tended to have lower than average M&R costs. It was anticipated that functionality would have an impact on M&R costs; however, the extent to which this would influence costs was not expected.

The remaining four newer buildings, denoted as "Non-standard " in the *Description* column, had varying initial construction costs (CA$/$m^2$) but three had higher than average combined M&R costs than the standard-design buildings. It was found that the higher costs were primarily the result of poor design quality or changes in requirements subsequent to the construction of the facility. Building Nos.13 & 14 required complete re-designs of their respective heating, ventilation and air conditioning (HVAC) systems. In the case of building 13, changes in requirements due to departmental restructuring and re-organisation were partially responsible for the high rehabilitation costs.

The difference in operating costs from base to base can also be seen in Table 3. The 15 administration buildings were found on six different military bases. Not only do the existing operating costs not take into account energy-related differences amongst individual buildings, but can be almost a two-fold difference in costs from one base to another. Rehabilitation costs were examined to see if the differences could be partially accounted for by the extent of individual building energy retrofits on each base. If this were true, then it would be expected that those bases with lower operating costs would have the highest percentage of energy retrofitted older buildings or newer energy-efficient buildings. This was not the case.

The results of heat loss calculations for typical buildings showed that a standard energy retrofit (re-roof, re-side and insulate, replace windows and doors) of an older building reduced heat loss by approximately 20%. A newly constructed building had approximately 40% less heat loss than a non-retrofitted building and 25% less heat loss than a retrofitted building. The increased demand for electricity from when the buildings were first constructed to the present however, ranged from 10% to 15%, depending upon the equipment and use of air conditioning. Based on these findings, collaboration with DND is ongoing to find a more accurate method of representing operating costs in the model.

**Table 3. Administration building annual equivalent costs (1999CA$) based on 40-year DSL**

| *Bldg* | *Description* | *Construction AEC* | *Operating AEC* | *Maintenance AEC* | *Rehabilitation AEC* | *Total AEC* |
|---|---|---|---|---|---|---|
| | | *1999CA$/m²* | *1999CA$/m²* | *1999CA$/m²* | *1999CA$/m²* | *1999CA$/m²* |
| 1 | Standard Army | 74.35 | 28.34 | 10.84 | 25.20 | 138.73 |
| 2 | Standard Army | 75.59 | 28.34 | 7.02 | 12.23 | 123.18 |
| 3 | Standard Army | 74.53 | 28.34 | 8.54 | 27.01 | 138.42 |
| 4 | Standard Army | 75.38 | 27.17 | 6.79 | 28.02 | 137.36 |
| 5 | Non-standard | 85.78 | 27.17 | 6.73 | 24.10 | 143.78 |
| 6 | Standard Army | 67.12 | 17.34 | 6.32 | 3.30 | 94.08 |
| 7 | Standard Army | 75.35 | 17.34 | 3.55 | 4.16 | 100.40 |
| 8 | Standard Army | 80.99 | 17.34 | 10.79 | 12.36 | 121.48 |
| 9 | Standard Army | 75.24 | 17.34 | 1.80 | 1.71 | 96.09 |
| 10 | Standard Army | 81.94 | 17.34 | 5.01 | 6.37 | 110.66 |
| 11 | Standard Army | 81.88 | 17.34 | 11.56 | 4.64 | 115.42 |
| 12 | Standard Army | 97.29 | 19.09 | 3.98 | 15.10 | 135.46 |
| 13 | Non-standard | 55.70 | 31.04 | 6.79 | 25.98 | 119.51 |
| 14 | Non-standard | 82.34 | 31.04 | 4.89 | 14.68 | 132.95 |
| 15 | Non-standard | 90.46 | 23.19 | 6.85 | 6.16 | 126.66 |

**Table 4. Annual equivalent M&R costs (1999CA$) based on first 20 years of service life**

| *Bldg* | *Description* | *Maintenance AEC* | *Rehabilitation AEC* | *Total AEC M&R* [1] |
|---|---|---|---|---|
| | | *1999CA$/m²* | *1999CA$/m²* | *1999CA$/m²* |
| 1 | Standard Army | 6.48 | 1.98 | 8.46 |
| 2 | Standard Army | 1.13 | 0.67 | 1.80 |
| 3 | Standard Army | 1.47 | 3.31 | 4.78 |
| 4 | Standard Army | 1.67 | 5.55 | 7.22 |
| 5 | Non-standard | 1.73 | 8.94 | 10.67 |
| 6 | Standard Army | 0.99 | 0.34 | 1.33 |
| 7 | Standard Army | 1.35 | 0.00 | 1.35 |
| 8 | Standard Army | 10.79 | 1.14 | 11.93 |
| 9 | Standard Army | 1.06 | 0.77 | 1.83 |
| 10 | Standard Army | 1.89 | 0.96 | 2.85 |
| 11 | Standard Army | 9.58 | 0.98 | 10.56 |
| 12 | Standard Army | 0.80 | 1.66 | 2.46 |
| 13 | Non-standard | 4.33 | 14.12 | 18.45 |
| 14 | Non-standard | 4.89 | 14.68 | 19.57 |
| 15 | Non-standard | 4.58 | 4.12 | 8.70 |

[1] AEC - annual equivalent costs calculated using equation (2) where N= 20 years, i = 5.6%

Annual equivalent costs were then re-calculated using the first 20 years of historical costs in order to compare M&R costs during the first half of each building's DSL. These results are shown in Table 4 above. The findings paralleled those of the 40-year DSL analysis. The most notable finding was the disparity in M&R costs between the majority of the standard design buildings and the newer infrastructure. While only four newer buildings are represented in this particular sample, the results are consistent with earlier findings presented to DND which found that newer buildings were shown to be incurring high maintenance costs early on in the DSL and were requiring significant rehabilitation work before approaching the middle range of the DSL (Newton, 2001). The high maintenance costs attributed to buildings 8 & 11 were a result of brickwork repairs and interior re-lamping.

## 4.2 Building QQ

An analysis of design and "as built" drawings, building inspection data, preventive maintenance data, maintenance and rehabilitation cost data and user perception surveys was done. Each building was then assigned a score, based on the seven quality dimensions, and a $Q_DQ$, $Q_CQ$, $Q_{OM}Q$ and QQ was calculated for all 15 buildings in the sample. As seen in Table 5 the standard-design buildings had QQ scores ranging from 0.81 to 0.82. The newer buildings had scores that ranged from 0.68 to 0.77. The preliminary findings indicated a possible trend in which design quality appears to be lower in the newer buildings. The lower QQ values were primarily a result of lower conformance and durability scores. Approximately 160 additional buildings remain to be assessed using the QQ formulae before solid conclusions can be drawn from the data.

**Table 5. Administration building quality quotient (QQ) scores**

| *Bldg* | *Description* | $Q_DQ$ | $Q_CQ$ | $Q_{OM}Q$ | *QQ* |
|---|---|---|---|---|---|
| 1 | Office - Standard Army | 0.82 | 0.80 | 0.73 | 0.82 |
| 2 | Office - Standard Army | 0.82 | 0.80 | 0.66 | 0.82 |
| 3 | Office - Standard Army | 0.82 | 0.80 | 0.80 | 0.82 |
| 4 | Office - Standard Army | 0.82 | 0.80 | 0.85 | 0.82 |
| 5 | Office - Non-standard | 0.80 | 0.60 | 0.73 | 0.77 |
| 6 | Office - Standard Army | 0.82 | 0.80 | 0.77 | 0.82 |
| 7 | Office - Standard Army | 0.82 | 0.80 | 0.69 | 0.81 |
| 8 | Office - Standard Army | 0.82 | 0.80 | 0.71 | 0.81 |
| 9 | Office - Standard Army | 0.82 | 0.80 | 0.69 | 0.81 |
| 10 | Office - Standard Army | 0.82 | 0.80 | 0.71 | 0.81 |
| 11 | Office - Standard Army | 0.82 | 0.80 | 0.75 | 0.82 |
| 12 | Office - Standard Army | 0.82 | 0.80 | 0.82 | 0.82 |
| 13 | Office - Non-standard, new | 0.66 | 0.60 | 0.58 | 0.68 |
| 14 | Office - Non-standard, new | 0.60 | 0.60 | N/A | N/A |
| 15 | Office - Non-standard | 0.76 | 0.60 | 0.65 | 0.73 |

## 4.3 Determining the impact of quality on life cycle costs

At the start of the research, it was hypothsised that an X-Y scatter plot, designating building life cycle cost as the dependent variable and the quality quotient as the independent variable could be created from the data and analysed to determine if a realtionship between the two variables could be quantified. Furthermore, it was proposed that the relationship would be assymptotic in nature and based on a power, polynomial or exponential model, similar to those shown in Fig. 2.

What has been found is that further analysis of operating costs is required before life cycle costs can be calculated. In addition, the impact of building functionality on M&R costs has proven to be of greater significance than was expected for the administation category of buildings. Thus, it may be necessary to consider the impact of building usage by modelling buildings not only by category but also by the extent to which usage influences M&R costs. Consideration must be given to a new model which considers function as an additional variable.

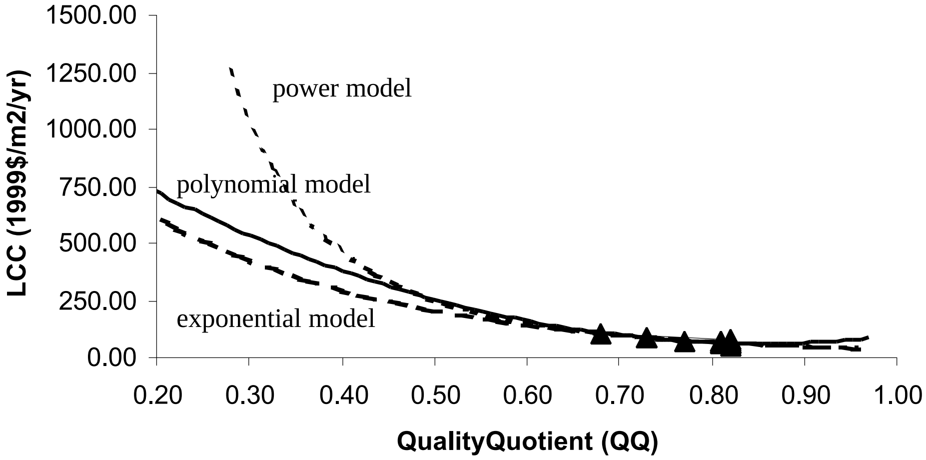


**Figure 2. Hypothesised comparison of LCC and QQ – exponential, polynomial and power models**

## 5 FUTURE RESEARCH

Work will continue on the analysis of the life cycle cost data and the determination of critical component service life for the remaining categories of facilities (residential, operational, maintenance garages, institutional and multi-use). The results from the heat loss calculations, load inventory and historical energy cost study will be used to determine a more accurate value for operating costs. New models will be examined that will include functionality as a variable. One option is to undertake an econometric analysis.

In addition to the theoretical determination of the QQ, the building and quality dimension analysis is currently undergoing a trial implementation with facility management staff at a nearby military base. Once this is completed, any recommended changes to the building and quality dimension analysis will be incorporated into the QQ metrics and the QQ values will be calculated for the remaining buildings.

## 6 CONCLUSIONS

The goal of the research was to determine the impact of quality on life cycle costs. A formula to measure building quality, called the quality quotient or QQ, was developed and then used to find a quality score for the sample buildings. These scores ranged from 0.6 to 0.82 with the older, standard- design buildings scoring the highest.

Total costs, based on actual service life and the first 20 years of service life, were calculated for the 15 buildings. Preliminary findings showed that, in the administration category of facilities, the non-standard designed buildings (newer) had incurred significantly higher rehabilitation costs in the first 20 years of service than the standard-design buildings. It was found that the operating costs currently used by DND do not accurately represent individual building operating costs, and that usage appears to play a greater role in influencing M&R costs than anticipated. Analysis of the remaining buildings will determine if the impact of usage is as significant in the five other categories of buildings as it is in the administrative category. Should this be the case, the proposed two-dimensional asymptotic model will need to be replaced with one that accounts for quality, cost and function.

## 7 ACKNOWLEDGMENTS

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# The Effect Of Reinforcement On The Risk Of Cracking in Hardening High Strength Concrete

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**Summary: High strength concrete (HSC) undergoes volume changes during hardening. These changes are generated by thermal effects and autogenous shrinkage and can lead to cracking if they are restrained. An engineering tool in order to control cracking is reinforcement. There are indications that the presence of reinforcement postpones the occurrence of major cracks at early age. This matter has now been investigated with the help of a Temperature Stress Testing Machine (TSTM). The reinforcement percentage (0%, 0.75%, 1.34%, 3.01%) and reinforcement configuration (1 bar and 4 bars) have been varied in a HSC with a water to cement ratio 0.33.**

**The results show that in specimens with four reinforcement bars the formation of microcracks increases the strain capacity of a concrete element before major cracks occur. Specimens with one reinforcement bar crack as sudden as plain concrete specimens. The influence of reinforcement percentage and configuration on the development of self-induced stresses is discussed. Furthermore the question is dealt with how the gain in strain capacity can be used for judging the risk of cracking. In this respect a strain criterion for designing durable HSC seems to be more suitable than a stress criterion and is formulated in this paper.**

**Keywords: High strength concrete, reinforcement, Temperature Stress Testing Machine, early-age cracking, cracking criterion**

## 1 INTRODUCTION

High strength concrete (HSC) has been developed for carrying higher loads than normal strength concrete. This is realized by applying lower water-cement ratios and admixtures. A low water-cement ratio leads to a denser microstructure, which makes HSC an ideal material for durable structures. However, HSC undergoes additional volume changes during hardening, so called autogenous shrinkage. Under restrained conditions this makes HSC even more prone to cracking in the early phase. With reinforcement this cracking can be controlled.

In order to enhance our understanding of the influence and effect of reinforcement on early-age cracking, an experimental research project has been conducted. Unreinforced and reinforced concrete specimens were tested in a Temperature Stress Testing Machine (TSTM). In this test device the concrete hardened under sealed conditions and the deformations were 100% restrained. Consequently stresses were generated in the specimen, which could lead to cracking when the tensile strength (or strain capacity) of the concrete was exceeded.

The results presented here refer to a HSC that was cured semi-adiabatically. Test variables were the reinforcement percentages and configurations. In parallel with the TSTM test the free deformation of plain and reinforced concrete was measured. From these measurements the effect of the reinforcement on the free deformation could be deduced. Furthermore, the development of the modulus of elasticity and the cube compressive strength were monitored.

The results indicated that a substantial gain in strain before the occurrence of major cracks can be achieved by applying reinforcement. In order to allow for this gain in strain capacity a stress-strain diagram reinforced concrete specimen in uniaxial tension is suggested. With the help of the proposed stress-strain relationship a strain criterion could be formulated for judging the risk of reinforced concrete at early age.

# 2 EXPERIMENTS

## 2.1 Programme

The stress development and the cracking behaviour of a reinforced concrete element was tested in a Temperature Stress Testing Machine (TSTM) with full restraint of the deformations. The free deformations of plain concrete and reinforced concrete were measured in dummies. Table 1 shows the concrete composition of the tested HSC (w/c ratio = 0.33). Specimens of each experiment were cast of the same batch of concrete and cured semi-adiabatically under sealed conditions.

**Table 1. Concrete composition per m$^3$**

| | |
|---|---|
| Water | 125.4 kg |
| CEM III/ B 42.5 LH HS | 237.0 kg |
| CEM I 52.5 R | 238.0 kg |
| Silicol SL (50/50 slurry) | 50.0 kg |
| Lignosulphonate | 0.9 kg |
| Naphtalene sulphonate | 9.5 kg |
| Gravel 4 - 16 mm | 973.5 kg |
| Sand 0 – 4 mm | 796.5 kg |

The TSTM specimens were tested with three different percentages of reinforcement, 0.75%, 1.34% and 3.02%, respectively, and without reinforcement. Each reinforcement percentage (ω) was realised with two different configurations of reinforcement bars (rebars): 4 ∅ 6 or 1 ∅ 12, 4 ∅ 8 or 1 ∅ 16, and, 4 ∅12 or 1 ∅ 25, respectively (Table 2).

**Table 2. List of experiments**

| *code* | *reinforcement configuration* | *reinforcement percentage (w)* |
|---|---|---|
| A | - | - |
| 4B | 4 ∅ 6 | 0.75 % |
| 1B | 1 ∅ 12 | 0.75 % |
| 4C | 4 ∅ 8 | 1.34 % |
| 1C | 1 ∅ 16 | 1.34 % |
| 4D | 4 ∅ 12 | 3.01 % |
| 1D | 1 ∅ 25 | 3.27 % |

## 2.2 Test equipment

The TSTM used for these experiments was used in combination with two dummies and cubes (Lura *et al*. 2001). One dummy was used to create reference conditions for the temperature development during hardening. In the second dummy the effect of reinforcement on the free deformation was tested. The cube compressive strength and the modulus of elasticity were tested at different early ages in order to monitor the development of concrete properties. Strain gages measured the self-induced stresses in reinforcement bars with a diameter of 16 mm and 25 mm.

### 2.2.1 The Autogenous Deformation Testing Machine (Dummy)

During hardening, the concrete temperature and the load-independent deformation were measured in an unrestrained specimen: the dummy (ADTM). The temperature was measured with thermocouples inserted in the concrete at different positions immediately after casting. So the temperature measurements could be started within approximately 30 minutes after mixing.

The load-independent deformations were measured with LVDT's. They measured the length changes of the dummies over a measuring length of 750 mm. This was done with the help of small steel bars perpendicular to the measuring direction. The bars were embedded in the specimen and passed the mould through holes (comparable to Figure 2).

2.2.2 The Temperature Stress Testing Machine (TSTM)

The TSTM is a horizontal steel frame in which hardening concrete specimens can be loaded in compression and in tension under various hardening conditions (Figure 2). Both load-controlled and deformation-controlled experiments can be performed under measured or prescribed thermal conditions.

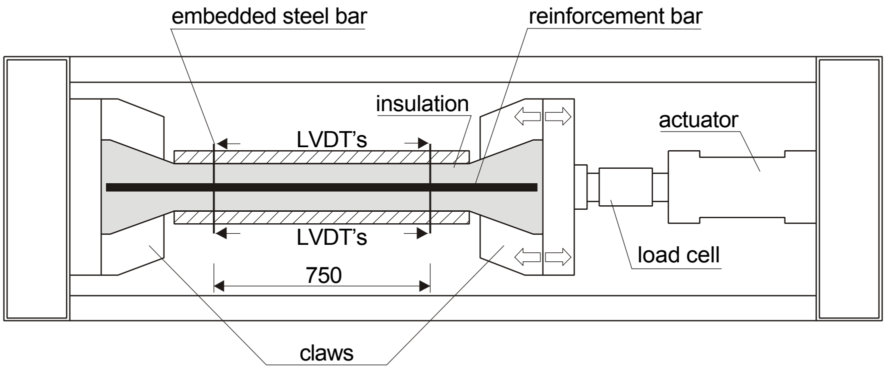



**Figure 2. The Temperature Stress Testing Machine (top view)**

To perform experiments in tension, a dovetailed interlock is used between the concrete specimen and the frame. Two steel claws hold the dovetailed specimen. One of the claws is fixed to the frame, the other is placed on roller-bearings and can be moved with the hydraulic actuator. Loads are measured with a load cell.

2.2.3 Strain gages

Strain gages measured the stresses generated in the reinforcement bar during the hardening phase. The gages and the connecting wires were placed in notches milled along the longitudinal welds. As the notches reduced the cross section considerably, particularly in case of small rebars, the strain was only measured in reinforcement bars with bar diameters of 16 mm and 25 mm.

Combined strain gages were chosen and placed opposite to each other in a cross section eliminating the effect of possible bending of the reinforcement in the measurements. One gage of this combination measured the deformation in longitudinal (Fig. 3, gage 1 and 3, respectively) and the other in transverse direction (Fig. 3, gage 2 and 4, respectively). Two of these combined gages were placed opposite to each other in one cross section forming a full bridge.

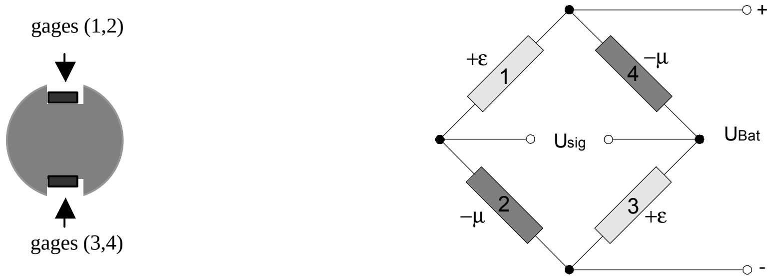
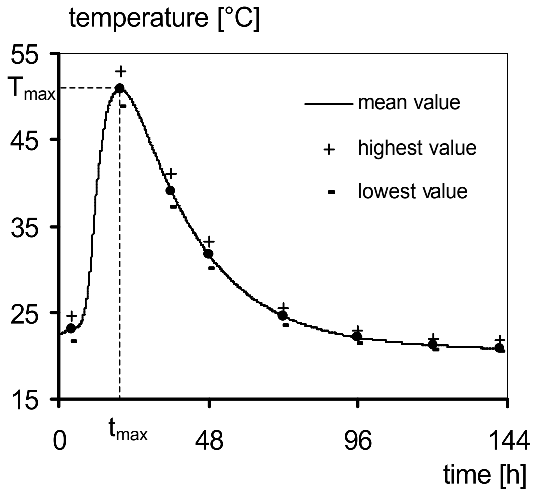



**Figure 3. Position of gages in the cross section of the reinforcement bar (left) and whole bridge formed by two combined gages (right)**

## 3 RESULTS

### 3.1 Temperature development

The heat evolution is an elegant parameter to which the development of concrete properties can be related (cube compressive strength, modulus of elasticity etc.). This heat evolution can easily be determined from the temperature curves during hydration with the help of an adequate maturity function (e.g. Arrhenius function). Thermocouples measured the temperature in all specimens of each experiment. In Fig. 4 the average temperature of all seven experiments is plotted. With a coefficient of thermal dilation $\alpha_T$, the thermal deformations $\varepsilon_T$ can be calculated with Eq. 1.




$$\varepsilon_T=\alpha_T\Delta T \qquad [1]$$

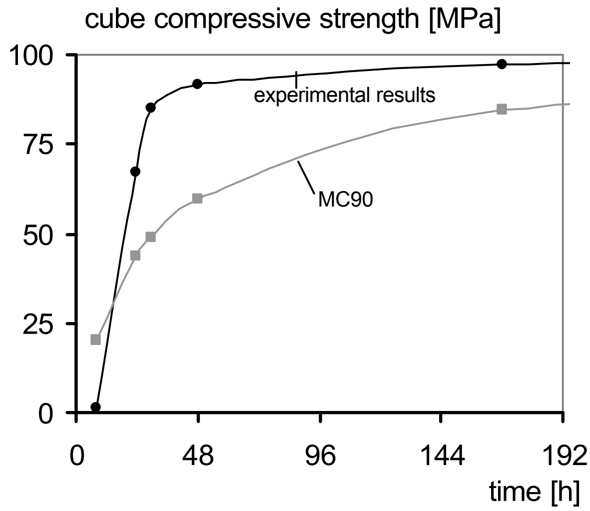
$\alpha_T$… coefficient of thermal dilation

$\Delta T$…temperature difference

**Figure 4. Average temperature development in the plain dummies of all seven experiments with scatter**

### 3.2 Development of cube compressive strength

The cube compressive strength was tested after different ages (8h, 24h, 30h, 48h, 168h and 672h). The experimental results are shown in Fig. 5. In addition, the cube compressive strength is plotted that was calculated according to MC90 (Eq. 2 and 3). As can be seen a rather poor agreement exists between the values calculated with MC90 and the experimental results. Obviously MC90 is not very accurate in case of HSC at very early age.




$$f_{cm}(t)=\beta_{cc}(t)f_{cm} \qquad [2]$$

$$ß_{cc}(t) = \exp\{s[1 \quad (\frac{28}{t})^{1/2}]\} \qquad [3]$$

$f_{cm}$…mean compressive strength after 28 days

$f_{cm}(t)$…mean compressive strength at the age of t days

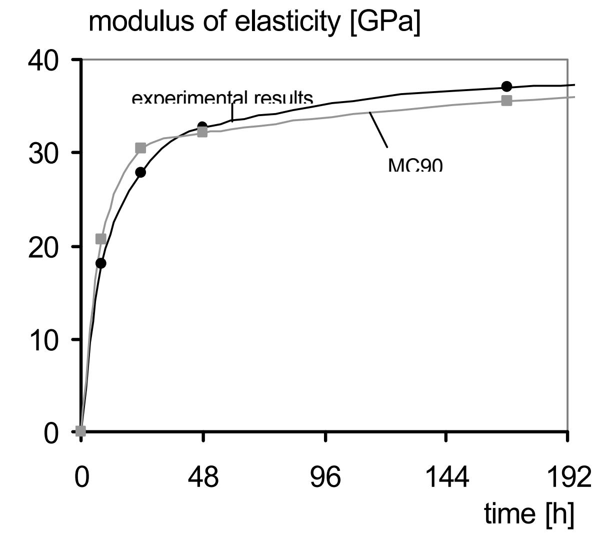
$\beta_{cc}$…coefficient depending on the age of concrete (t)

s…parameter, depending on the type of cement (0.2)

**Figure 5. Cube compressive strength tested in all experiments and calculated according to MC90**

### 3.3 Development of modulus of elasticity

The modulus of elasticity has been tested after 1 day, 3 days, 7 days and 28 days. As for the cube compressive strength, these results and the results found with the MC90 (Eq. 4 and 5) are shown in Fig. 6. Whereas the results of the cube compressive strength according to MC90 did not fit the experimental results, the tested modulus of elasticity could be well approximated with the help of Eqs. 4 and 5.




$$E_{ci}(t) = \beta_E(t)\, E_{ci} \qquad [4]$$

$$\beta_E(t) = [\beta_{cc}(t)]^{1/2} \qquad [5]$$

$E_{ci}$…modulus of elasticity after 28 days

$E_{ci}(t)$…modulus of elasticity at the age of t days

$\beta_E$…coefficient depending on the age of concrete (t)

**Figure 6. Development of modulus of elasticity from experiments and calculated according to MC90**

### 3.4 Development of free deformation

In all experiments the free deformation has been measured in a plain and a reinforced dummy specimen. As the coefficient of thermal dilation $\alpha_T$ is an intrinsic material property of the tested HSC and the temperature development of all specimens was about the same (Fig. 4) the deformations measured in all seven plain specimens is also nearly the same (Fig. 7, left). The effect of reinforcement on the free deformation can be seen in Fig. 7, right. As a first approximation the free deformation $\varepsilon_c(t)$ of concrete can be calculated as follows

$$\varepsilon_c(t)=\Sigma\Delta\varepsilon \qquad [6]$$

$$\Delta\varepsilon=\Delta\varepsilon_T(T)+\Delta\varepsilon_a(\alpha,T) \qquad [7]$$

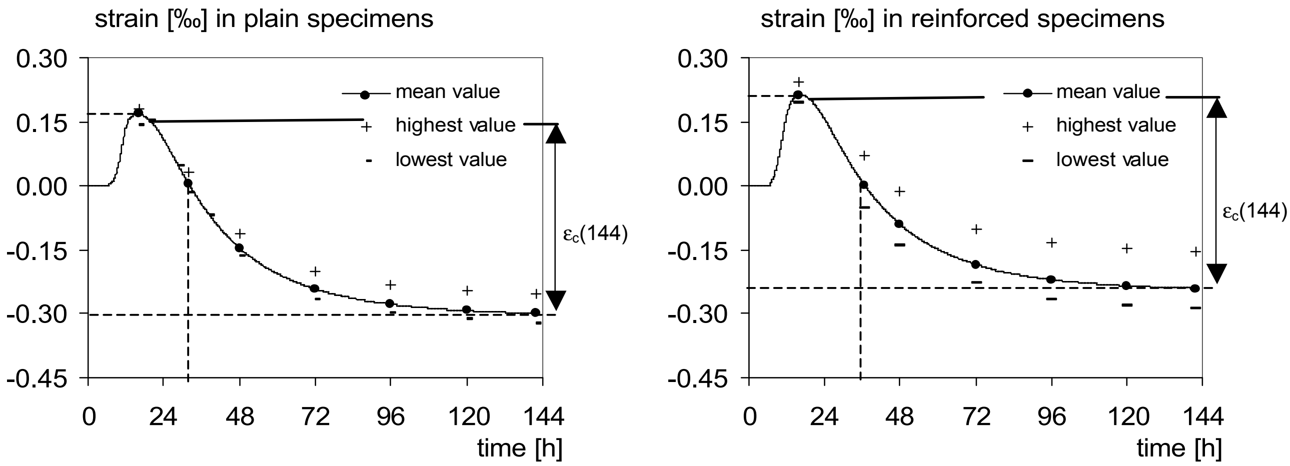



**Figure 7. Mean value of free deformation measured in all experiments with scatter on plain dummies (left) and reinforced dummies (right), HSC cured semi-adiabatically**

$\Delta\varepsilon_T(T)$…thermal strain increment depending on the temperature T

$\Delta\varepsilon_a(\alpha,T)$.autogenous strain increment depending on the temperature T and the degree of hydration $\alpha$
(Note: in fact autogenous deformations are also temperature dependent!)

There are three main differences between the measurements in plain and in reinforced dummy specimens. Firstly, reinforced specimens expand more. Secondly, reinforced specimens pass the zero strain line later. Thirdly, measurements in reinforced specimens vary much more than in plain specimens (due to different reinforcement percentages and configurations).

As expected the total free deformation measured after 144 hours in reinforced specimens, i.c. $\varepsilon_c(144)=0.455*10^{-6}$, is smaller than in plain specimens ($\varepsilon_c(144)=0.468*10^{-6}$). On average, reinforcement restrains the free deformation of the tested HSC by 2.8 %. It thus appears that the presence of reinforcement does not to influence the free deformation of the samples very much.

### 3.5 Stress development in TSTM

Table 3 shows the measured stress values for all experiments. It appears that the compressive stresses $\sigma_{D,total}$ (i.e. the force measured in the TSTM divided by specimen's cross section) at time $t_D$ when the compressive stress is maximal, increases with increasing reinforcement percentage (see also Figure 8, left). As a consequence the moment $t_{02}$ at which the stress development passes the zero-stress state in the cooling phase is postponed. In specimens with four reinforcement bars microcracks form before the first through-crack. This is indicated by a bending curve. In specimens with the smallest bar diameter (6 mm) the yield stress of the reinforcement was exceeded immediately after cracking of the concrete. Consequently this specimen failed as a plain specimen. In any case, specimens with one reinforcement bar cracked in the same sudden way as plain concrete specimens ($t_{el} \approx t_{cr}$, Table 4).

**Table 3. measured stress values for all experiments**

| *code* | $t_{01}$ | $t_D$ / $s_{D,total}$ | $t_{02}$ | $t_{el}$ | $t_{cr}$ / $s_{cr,total}$ |
|---|---|---|---|---|---|
| A | 6.5 | 13.7 / -0.98 | 19.6 | 28.7 | 28.7 / 3.38 |
| 4B | 6.0 | 15.1 / -1.6 | 22.5 | 32.1 | 32.1 / 3.47 |
| 1B | 6.5 | 14.7 / -1.6 | 22.5 | 27.4 | 32.1 / 3.57 |
| 4C | 6.3 | 15.4 / -1.94 | 25.6 | 35 | 38.4 / 3.40 |
| 1C | 7.4 | 14.6 / -1.72 | 21.4 | 27.2 | 27.4 / 3.33 |
| 4D | 8.0 | 16.1 / -2.90 | 26.0 | 30 | 34.9 / 3.26 |
| 1D | 8.3 | 15.0 / -2.53 | 24.4 | 29.7 | 29.7 / 2.67 |

*t [h], s [MPa]refering to $A_{total}$=15000 mm²*

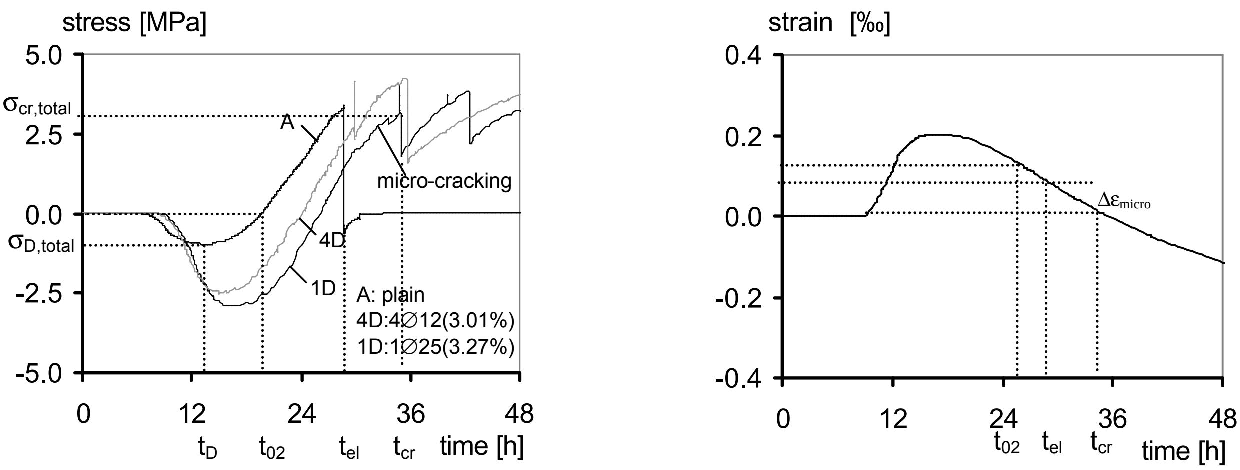



**Figure 8. Influence of reinforcement percentage and configuration on the stress development and the resulting gain in strain (De$_{micro}$) in specimen 4D (4Æ12)**

#### 3.5.1 Stress-inducing strain

The stress-inducing strain is given for all experiments in Tab. 4. These values have been measured in the plain dummy of each experiment and they are defined as in Eqs. 8-10. The "strain gain" $\Delta\varepsilon_{micro}$, i.e. the strain measured from the moment that the stress-strain curve started to flatten until cracking, is highest in the TSTM specimen of experiment 1B. Despite this single result the common trend of the results on the strain gain $\Delta\varepsilon_{micro}$ is that $\Delta\varepsilon_{micro}$ is about ten to twenty times bigger in specimens reinforced with four rebars compared with specimens reinforced with one rebar. For both reinforcement configurations it can be said that the post-elastic strain gain increases with the reinforcement percentage.

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$$\varepsilon_{el} = \varepsilon_c(t_{el}) - \varepsilon_c(t_{02}) \qquad [8]$$

$$\varepsilon_{cr} = \varepsilon_c(t_{cr}) - \varepsilon_c(t_{02}) \qquad [9]$$

$$\Delta\varepsilon_{micro} = \varepsilon_{cr} - \varepsilon_{el} \qquad [10]$$

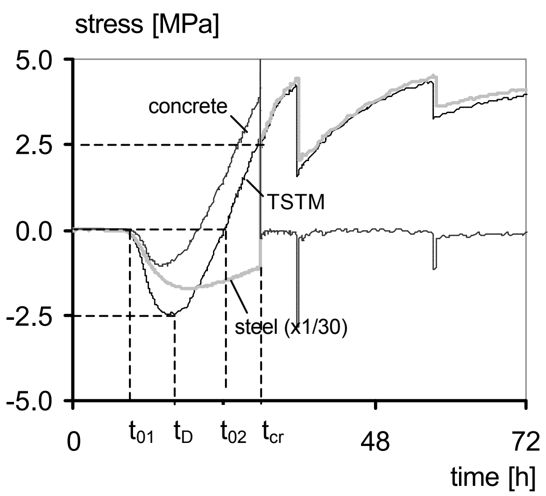
**Table 4. measured strain values for all experiments**

| *code* | *w* | $e_{el}$ | $e_{cr}$ | $De_{micro}$ | $h_{cr}$ |
|---|---|---|---|---|---|
| A | - | 0.091 | 0.091 | 0.000 | 1.000 |
| 4B | 0.75 | 0.108 | 0.108 | 0.000 | 1.187 |
| 1B | 0.75 | 0.069 | 0.135 | 0.066 | 1.484 |
| 4C | 1.34 | 0.103 | 0.134 | 0.031 | 1.473 |
| 1C | 1.34 | 0.084 | 0.087 | 0.003 | 0.956 |
| 4D | 3.01 | 0.057 | 0.122 | 0.065 | 1.341 |
| 1D | 3.27 | 0.069 | 0.083 | 0.014 | 0.912 |

*reinforcement percentage w[%], e [‰],* $h_{cr}$ *[-]*

### 3.5.2 Stresses in concrete and in reinforcement

In the previous section the effect of reinforcement on the stress and strain development in composite specimens is shown. Before cracking the stress in the concrete is fully dependent on the imposed strain, irrespective of the amount of reinforcement. Figure 9 shows the stresses in the concrete, the steel and the composite cross section of specimen 1D prior and after cracking. After the first crack occurred the stress in the concrete at the location of the crack is about zero. The further performance of the specimen is determined by the bond properties between steel and concrete.




$$s_c = \frac{s_{total} \quad A_{total} \quad s_s \quad A_s}{A_c} \qquad [11]$$

**Figure 9. Stress development in TSTM specimen, in the cast-in reinforcement bar of 25 mm diameter and in the concrete cross section of test 1D**

The stresses in the rebar ($\sigma_s$) were calculated from the measured strain (see strain gages). In order to fit into the figure they were plotted with a factor 1/30. The strain gage happened to be where the specimen cracked for the first time. The concrete stresses in the concrete cross section ($A_c$=$A_{total}$-$A_s$) were calculated with Eq. 11.

## 4 ADDITIONAL STRAIN CAPACITY PRIOR TO MAJOR CRACKS

From the experimental results it could be seen that the strain capacity of specimens reinforced with 4 rebars is considerably increased caused by the formation of microcracks. It seems to be appropriate, therefore, to formulate a strain criterion in order to judge the risk of cracking at early age.

Different researchers proposed diagrams for the stress-strain behaviour of early age concrete under tension. In normal strength concrete (NSC) a non-linearity can be found at approximately 40% of the failure load (Larson 2000). Hedlund (1996) found a

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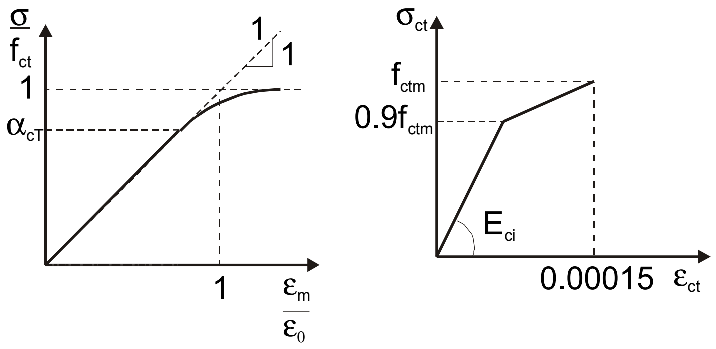
higher level for high performance concrete. In order to account for the non-linear stress-strain behaviour at high tensile stresses Jonasson (1994) and Hedlund (1996) developed the stress-strain relation shown in Figure 10, left. The MC90 handles for 28 days old concrete the diagram shown in Figure 10, right.




$f_{ct}$,$f_{ctm}$…tensile strength

$\alpha_{cT}$…relative stress level above which non-linear stress-strain behaviour is present [-]

$\varepsilon_m$… material strain

$\varepsilon_0$… fictitious strain linearly related to the tensile strength, $f_{ct}/E_c$

**Figure 10. Non-linear stress-strain behaviour under tension according to Jonasson (1994) and Hedlund (1996), left and stress-strain diagram for uniaxial tension according to MC 90, right**

In our experiments we did not find any non-linearity in plain concrete specimens. One reason might be the high stiffness of the tested concrete mixture. Another reason might be the long measuring length (Hordijk 1991) resulting in snap back behaviour. However, in experiments on specimens with 4 reinforcement bars it was observed that the stress-strain curve became flatter at high stress levels. This flattening could be explained by the formation of microcracks. These microcracks can result in an additional strain $\Delta\varepsilon_{micro}$. The values of $\Delta\varepsilon_{micro}$ can be taken from Tab. 4. According to Lokhorst (1998) plain concrete specimens crack at 0.75 times the actual tensile splitting strength ($\sigma_{cr}=f_{ct,sp}(t_{cr})$) at early age. This was taken into account when adjusting the stress-strain diagram of MC90 for a stress-strain diagram for early-age reinforced concrete (Figure 11). In this diagram the value of $\Delta\varepsilon_{micro}$ depends on the reinforcement ratio and configuration of the rebars in the cross section (see next section).

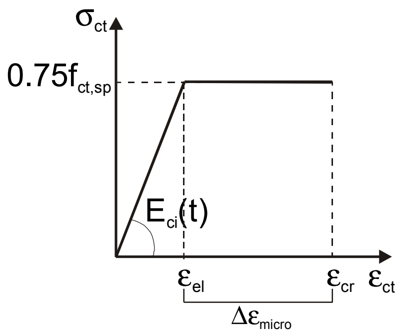



**Figure 11. Stress-strain diagram for early age reinforced concrete under tension**

The reason why more microcracks form in specimens with four reinforcement bars is illustrated in Fig. 12. Cracks always start in the weakest part of the specimen. The surface zones, especially the corners of the specimen, are likely to be the weakest parts of the cross section. From there the crack grows to the centre of the specimen. In specimens with four reinforcement bars a crack growing from the corner will meet a reinforcement bar in an early stage of crack growth. Thanks to the reinforcement bar the stresses are redistributed and more microcracks are generated before the specimen really cracks (major cracks). The sum of the microcracks' widths $\Sigma w_{micro}$ constitute the additional strain capacity $\Delta\varepsilon_{micro}$ of the reinforced concrete specimen.

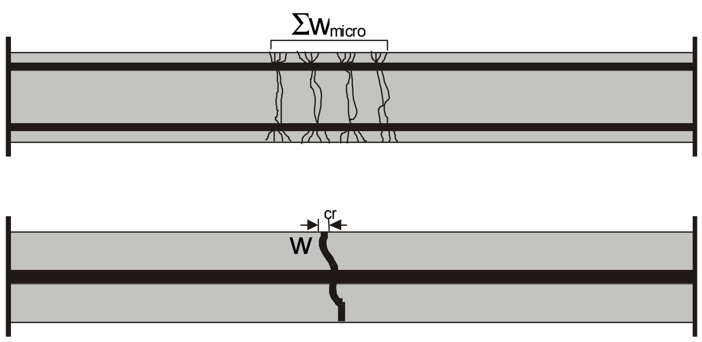



**Figure 12. Cracking in specimens with 4 rebars and 1 rebar (longitudinal and cross section)**

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# 5 CRACK CRITERION

In literature (Rostasy 2000, Larson 2000) different criterions are used in order to quantify the risk of cracking in early age concrete. In plain concrete a stress criterion is mostly applied, relating the stress at the moment of cracking to the tensile splitting strength at the same moment (e.g. Lokhorst 1998). Our results show that due to reinforcement microcracks can form which enhance the strain capacity of concrete before major cracks occur. Therefore a “strain enhancement factor”, $\eta_{cr}$, is introduced in order to indicate the proneness to cracking of reinforced specimens compared with plain concrete specimens (Eq. 12), viz.:

$$\eta_{cr} = \frac{\varepsilon_{cr,reinforced}}{\varepsilon_{cr,plain}} \qquad [12]$$

$\varepsilon_{cr,reinforced}$ and $\varepsilon_{cr,plain}$ are the stress-inducing strains until the moment that major cracks are formed in a reinforced specimen and a plain specimen, respectively. In this study the value of $\varepsilon_{cr,plain}$ is to be taken from Table 4, i.e. $\varepsilon_{cr,plain} = \varepsilon_{el} = 0.091\ 10^{-3}$ (specimen A). The strain enhancement factor depends on the reinforcement percentage and configuration and has been calculated for all experiments in Table 4. The results show that the strain enhancement factor increases with increasing reinforcement percentage. Even more important appear to be the reinforcement configuration: four reinforcement bars are much more effective than only one bar.

It is noticed that the value of $\varepsilon_{cr,plain} = 0.091\ 10^{-3}$ is based on only one test. This might be the reason why the strain enhancement factor $\eta_{cr}$ for the specimen with only one rebar are below one because of a relatively high value of $\varepsilon_{cr,plain}$.

# 6 DISCUSSION

The effect of reinforcement on the risk of cracking has been investigated on seven single experiments. In order to verify trends more experiments are to be performed. Nevertheless the following trends are discussed below not considering the lowest reinforcement percentage (series B).

## 6.1 Restraining free deformation

The experimental results on the dummies show that reinforcement restrains the free deformation. Assuming that the coefficient of thermal dilation of steel and concrete is about the same, the thermal shrinkage is not restrained. Therefore reinforcement has only an effect on autogenous shrinkage. This was also found by Sule et al. (2000a) in isothermal tests on different temperatures. Due to their bigger specific surface and their distribution in the cross section, four rebars restrain the autogenous shrinkage more effectively than one rebar.

## 6.2 Postponing the moment of stress free state $t_{02}$

The results summarized in Table 3 show that in semi-adiabatic tests the compressive stress ($\sigma_{D,total}=F_{TSTM}/A_{total}$) increases with increasing reinforcement percentage. When separating concrete and steel stresses (as in Fig. 9) it becomes obvious that in reinforced specimens only the steel stress increases while the concrete stress stays nearly the same as in plain specimens. However the compressive stresses $\sigma_{D,total}$ increase with the reinforcement percentage and the moment $t_{02}$ at which the composite specimen passes the stress-free state is postponed ($t_{02}$: transition from compression to tension).

Emborg & Bernander (1994) consider the mean temperature $T(t_{02})$ of the cross section at $t_{02}$ as a crucial parameter when calculating thermal stresses. The later the zero stress is passed, the smaller is the remaining imposed strain in the cooling phase that has still to be compared with the ultimate failure strain for judging the risk of cracking. So, the later the zero stress is passed, i.e. the lower the zero stress temperature, the lower the risk of cracking.

In the case of reinforced specimens, however, postponing the moment when the stress-free state is passed does not decrease the risk of cracking. The stresses measured in the TSTM refer to the composite cross section. This means that when the stresses measured in the TSTM specimen passes the stress-free state ($t_{02}$) the concrete is already under tension while the reinforcement bar is under compression (Fig. 9). In this case the “overall zero stress” is not a good indicator of the remaining strain capacity of the concrete.

## 6.3 Increasing the strain capacity

With increasing reinforcement percentage the strain capacity prior to major cracks increases. This increase is only very small for specimens with one reinforcement bar as they show about the same sudden cracking behaviour as plain specimens. Specimens with four reinforcement bars, however, show a substantial increase in strain capacity before the first through-crack. This is supposed to be due to microcracking. The increase in strain capacity can reach values up to 0.000065 (4D), which is 44% of the concrete tensile strain of plain concrete at an age of 28 days (Fig. 10 right).

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### 6.4 Judging the increase of strain capacity

Judging the increase of strain capacity with help of the strain enhancement factor (Eq. 12) reveals that specimens reinforced with one rebar perform less advantageous than plain specimens. In specimens with four rebars the strain capacity was found to increase substantially, i.e. by up to 48%. Although the strain gain $\Delta\varepsilon_{micro}$ is bigger in 4D (higher reinforcement percentage) than in 4C the total strain $\varepsilon_{cr}$ at the moment of cracking is lower. The reason is that the formation of microcracks start very early and consequently the value of $\varepsilon_{el}$ is relatively small.

### 6.5 Discussing microcracks in view of durability

Figure 12 illustrates how microcracks form in specimens with four rebars, increasing the strain capacity of these specimens. In view of durability it is of primary importance to know if these microcracks impair the quality of the concrete cover. Via a permeable surface, water can reach the interconnected network of pores, microcracks and macrocracks. As water is the medium that plays a very important role in deterioration processes, such as sulphate attack, alkali-silica expansion and frost action, the high quality of the surface is indispensable in order to provide reliable long-term performance, especially when exposed to aggressive environments.

Little information is available about the effect of microcracks in real sructures on the durability. It is still a matter of spectaculation to which extent these microcracks may close due to self healing. Sometimes surface cracks are compressed later on in the hydration process. Sule and van Breugel (2000b) investigated how these compressed cracks performed in a TSTM without the presence of water. They found that only very small cracks close again so that the specimen regains the strength to bear loads applied after a period of compression. Based on theoretical and experimental research Edvardsen (1999) investigated the effect of crack healing in structures subjected to water-pressure loads. She proposes permissible crack widths, which can be expected to obtain an almost total self- healing after a few weeks of water pressure exposure. Therefore it is important to control the crack width with reinforcement.

## 7 CONLUSION

The experimental results (tests C and D) on reinforced TSTM specimens show that microcracks form before major cracks occur. As the formation of microcracks results in a gain in strain capacity, a strain enhancement factor has been formulated in order to judge the effect of reinforcement. It was found that specimens with four reinforcing bar performed better than plain specimens and specimen with only one bar. In specimens with 4 rebars the moment of the first through-crack could be postponed substantially: an increase in the strain capacity up to 48% has been found. In order to control the crack width in view of durability in early-age concrete further research is necessary.

## 8 ACKNOWLEDGMENTS

The assistance of the laboratory technicians of Stevin laboratory is gratefully acknowledged. A special thanks goes to Mr A. van Rhijn for performing the experiments. This project can be realised thanks to the financial support of the Dutch Technology Foundation (STW) and the Ministry of Transport, Public Works and Water Management (RWS).

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# Quantifying The Likelihood Of Failure For Housing Stock

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**Summary: The likelihood of failure for a dwelling was an essential element of a condition index developed for the Queensland Department of Housing (DOH) by CSIRO, QUT and DOH jointly. This paper describes the steps taken and method used to derive a suitable estimator for this element. In particular it focuses on the preliminary statistical analysis of a sample from the 55,000 dwelling and the findings of this analysis. The objectives of the analysis were to identify key characteristics of the housing stock and to extract a relationship between the dwelling age profile and projected maintenance requirements. The analysis revealed the single factor that has the greatest influence on the cost of repairs of a dwelling was the dwellings age and ascertained that the higher repair costs for older dwellings is due to the number of repairs needed which increases with age. From this analysis an appropriate survival distribution to predict the probability of the properties survival beyond the maintenance horizon was extracted.**

**Keywords: Condition index; statistical analysis; survival distribution.**

## 1 INTRODUCTION

The Queensland Department of Housing has developed a Property Standard Index (PSI) in collaboration with CSIRO – QUT Construction Research Alliance to summarise the overall condition of the building stock of DOH. The Property Condition Index was developed to assist DOH in its decisions on selling, maintaining and reviewing maintenance strategies of its dwelling stock.

The model developed was based on the Housing Stock Condition Index (HSCI) developed by an earlier CSIRO study (Tucker et al (1996)) and modified to be consistent with the needs of Queensland DOH. The model utilised data collected identifying maintenance needs of a property, both current needs and an indication of those deemed necessary over the next ten years. Further to this, information on the standard of the property was collected to aid in understanding how well the properties match with the current standards required of the DOH housing stock. This data was then supplemented with forecast of future maintenance needs beyond the ten-year horizon to provide an indicator to be assessed when considering the future of a property. It was the estimation of the forecast maintenance needs that posed the greatest challenge and was the catalyst for this analysis.

This paper relates the process of analysis undertaken to uncover key characteristics of Queensland's public sector housing stock in an effort to extract a relationship between housing types and maintenance requirements. The analysis performed utilised a large data set that, at the time of the analysis, was still in the process of being collated. The complete database comprised of data items from a portfolio of 55,000 housing units with 632 variables for each housing unit, a total approaching 35 million individual data entries.

## 2 DATA

For this analysis data a sample from the portfolio of 55,000 dwelling units was used. The data was collected by Queenslands Department of Housing during its Property Condition Appraisal Project (PCAP). PCAP provided a detailed and accurate snapshot of the housing portfolio. In all, 632 fields of data items were collected for each property. These fields included descriptive information on the dwellings and their elements, a 0-10 years replacement rating for the elements, and for the maintenance identified for the years 0-3, a replacement cost. The maintenance costs estimated by the inspectors were determined on the extent of repairs needed and were inclusive of any extra costs due to regional cost differences.

The analysis was undertaken on a sample of approximately 6500 detached houses and all associated data was extracted from the comprehensive database assembled. The data was collated and then entered into a Microsoft Access© database. The statistical analysis was completed using SAS©, Splus© and SPSS© statistical software as well as Microsoft Excel©.

## 3 METHODOLOGY

Exploratory data analysis (EDA) and data visualization techniques were the starting points of the analysis, chiefly to identify possible relationships in the data. The visual techniques used include box plots and histograms of individual variables which aid in understanding the distributional properties of the data and scatter plots of pairs of variables to identify possible relationships. This analysis sought to determine key characteristics of the data that enabled the observations to be classified into groups. To achieve the classifications many of the descriptive variables gathered were utilised and the data subjected to analysis of variance techniques. The ANOVA procedure formed an integral part of the overall analysis. The ANOVA is a statistical technique which tests differences between two or more groups by comparing the variation between the groups with the variation within them. In an ANOVA, the F-ratio is the statistic used to test the hypothesis that the means of groups within the variables are significantly different from one another Care was taken not to under or over analyse the data as, if it was dissected into small enough sections significant differences will be found between groups simply by chance. Similarly, under analysing the data risks missing significant differences or interpreting incorrect results due to confounding. The one-way ANOVA was used to test the differences of the means between the groups and when a statistically significant value resulted from the ANOVA a Post-Hoc Comparisons of Means was performed to determine which groups were significantly different from each other (Einot, 1975). To measure the mean differences between groups the effect-size statistic was used (Cohen, 1990). This statistic, which measures the magnitude of the difference between the means of two groups, is calculated by taking the difference in the mean for the groups and dividing it by the average standard deviation for the two groups. The resulting number represents how many standard deviations the two groups differ by and this provides a measure of the true significance.

Given that we are trying to understand how properties perform over time in relation to their maintenance requirements, this analysis should have been completed using longitudinal data but in the absence of this data, the analysis was carried out using the data available. One of the concerns when converting cross-sectional data to longitudinal data is the assumption that the newer properties will become like the older properties as they age. In reality, with changes in techniques and materials (improvements or declines) and the differences in craftsmanship, the likelihood is that they will differ. In view of the above concerns, caution was exercised in any decision to manipulate the current data sets and the interpretation of the results.

## 4 DATA ANALYSIS

This analysis was undertaken specifically to uncover key characteristics of the housing stock and to extract a relationship between the dwelling age profile and projected maintenance requirements to provide the basis to develop a suitable weighting scheme that would be utilised as part of an algorithm to rate the condition of a property.

From the exploratory data analysis descriptive information from the sample data set was collated. In the sample selected the year of construction of the properties ranged from 1930 to 1999, which reflected the distribution of the population, and each of the variety of claddings and roof types were adequately represented. The distribution of the age of the properties in the sample is illustrated in Figure 1. Assessing the chart it can be seen that there were periods of relatively constant rate of increase for the number of detached houses in the portfolio, which increased over time. There are however three distinct peaks of construction activity in the periods 1950, 1970/75 and 1985/1990. These peak construction periods are likely to result in an upsurge in maintenance requirements as the number of properties whose elements near the end of their expected lifecycle increase. There is also an apparent decline in the number of properties constructed in the past 8 years. This has not been investigated although may be due to the increased development of multi-unit dwellings as opposed to detached houses being analysed here.

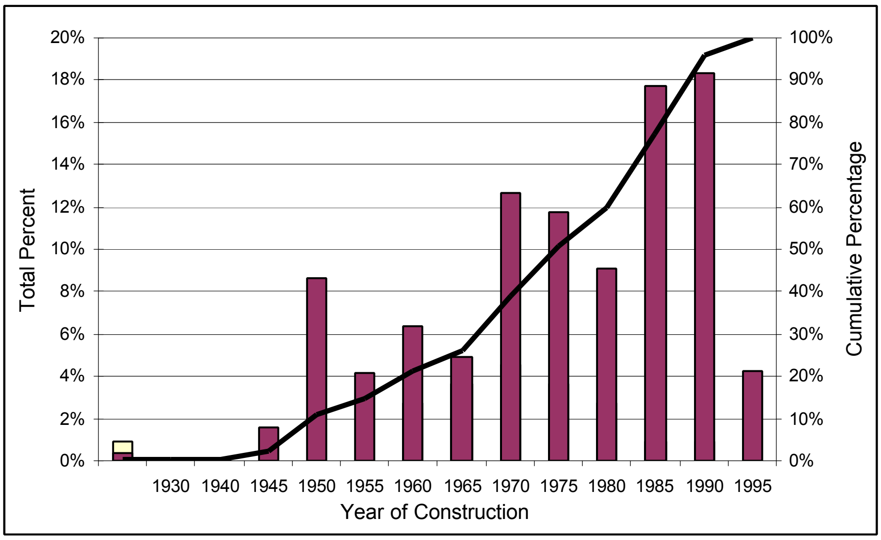


**Figure 1: Distribution of the age of the sample dwellings**

The chart shows that 40% of the current stock is less than twenty year old and about 35% are older than 35 year old. The age distribution is further analysed later but this chart underlines the importance of having a measure of property condition as the properties around 20 year old are likely to be due for upgrades while properties older than 35 may soon be in need of major maintenance and with reducing maintenance budgets early identification of these properties will assist in developing a planned maintenance strategy to ensure the maintenance needs can be met and the maximum utility is gained from each dollar.

Maintenance is required on a property due to component failure of some form. Some of the likely reasons for maintenance/replacement of a component speculated include:

- Health and Safety – the component requires replacement to eliminate the risk of injury
- Physical – the component has deteriorated and the physical collapse is imminent
- Economic – no longer economic to maintain
- Functional – the component no longer functions for the purpose it was originally intended
- Technological – advances in technology had rendered the component outdated and replacement is demanded as a result
- Social – the component is no longer socially accepted
- Legal – the component is legally required to be replaced

Using this list of possible reasons for maintenance, speculation on suitable variables that indicate failure can be done. Obviously the variables need to be from the variable list within the dataset. Time was expected to be a significant factor leading to failure and could be aided by the environment the property is in. This cause could lead to many of the reasons outlined above and the risk could vary depending on the materials of construction and other component within a dwelling. Thus the major variables expected to be help identify dwellings with high maintenance requirements are age of the house and spatial location, the later being a latent variable for the dwelling environment such as climatic conditions. Also, as it was thought the maintenance costs for the properties may vary dependent on the material of construction such as cladding type and roofing material and floor type, i.e. slab on ground, low set, high set etc. These characteristics were treated indicators and used as factors to be investigated during the analysis.

There were many forms of cladding and roofing materials that the houses in the sample were constructed from, although predominantly consisted of three combinations:

- Brick veneer clad and tiled roof,
- Brick veneer clad and sheet roof, and
- Timber clad and sheet roof.

Table 1 below displays the marginal distribution of properties in the sample constructed in the various combinations of materials used in the portfolio. The portfolio consists of properties throughout Queensland, which, for administrative purposes is divided into 18 area offices. These area offices were used as indicator variables for the property location in the analysis. The cladding types are spread proportionally throughout these regions although Northern Queensland has a greater proportion of houses clad in concrete block. The significant proportions are in bold type.

**Table 1 Proportion of dwellings in each construction materials category**

| | Roofing Material | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Cladding | AC Sheet | Alum | Colorbond | Concrete Tile | Painted Gal sheet | Unpainted Gal sheet | Terracotta Tile | Grand Total |
| AC Plank | 0.05% | | 0.39% | 0.79% | 0.42% | **4.45%** | 0.05% | **6.13%** |
| AC Sheet | 0.29% | 0.06% | 1.36% | 1.00% | 1.11% | **6.07%** | 0.22% | **10.10%** |
| Brick/AC | | 0.02% | 0.02% | 0.54% | | 0.29% | 0.03% | 0.89% |
| Brick/AC Plank | | | 0.11% | 0.37% | | 0.51% | | 0.99% |
| Brick/FC Plank | | | 1.85% | 0.52% | | 0.15% | | 2.53% |
| Brick Veneer | 0.17% | 0.02% | **13.57%** | **23.90%** | 0.20% | 0.48% | 0.92% | **39.26%** |
| Concrete Block | 0.03% | 0.02% | **4.19%** | 0.28% | 0.09% | 1.03% | | **5.64%** |
| Concrete | 0.28% | | 0.02% | 1.62% | 0.35% | 1.49% | 0.37% | **4.13%** |
| FC Plank | | | 0.94% | 0.15% | 0.03% | 0.55% | | 1.68% |
| Rendered Masonry | 0.17% | | 0.31% | 2.90% | 0.26% | 0.22% | 0.39% | **4.24%** |
| Timber | 0.52% | 1.05% | **5.51%** | 2.49% | **7.33%** | **6.92%** | 0.60% | **24.43%** |
| Grand Total | 1.51% | 1.16% | **28.25%** | **34.56%** | **9.80%** | **22.16%** | 2.57% | **100.00%** |

The three major combinations represent 80% of the portfolio and when AC clad properties are included into the Timber classification these categories represent 95% of the population. Table 1 shows that the Brick Veneer/Concrete Tile category covers nearly a quarter of the sample and Brick Veneer clad properties represent almost 40% of all properties. The next largest classification is Timber clad/sheet roofing representing 20% of the sample. Combining the age distribution with the materials classifications reveals how the portfolio has transformed over time. Figure 2 illustrates the changing trend in cladding materials used over time.

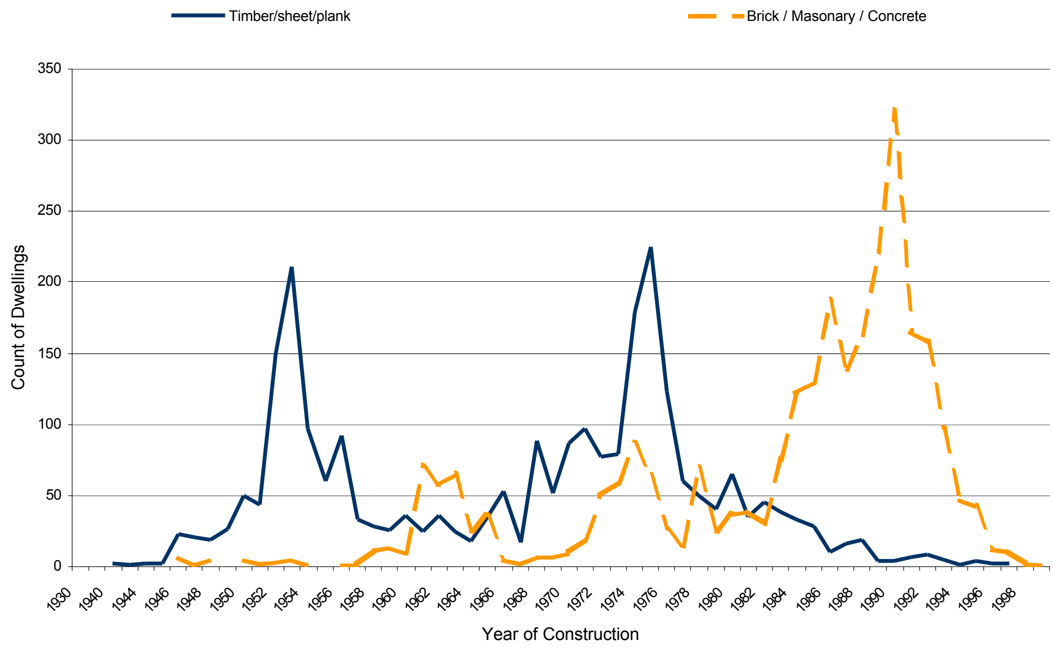


**Figure 2: Changing trend in construction materials of the outer walls.**

Over the past 20 years the DOH has built many of its new houses using brick as the main construction material. One of the many benefits of constructing brick veneer houses is minimal external painting for the dwelling. External painting is one of the major maintenance costs for portfolio managers. With approximately 55000 properties in their portfolio and an expected repaint term of between 8 and 10 years, up to 7000 properties per year could require external painting if the age distribution of the portfolio was uniform. However, as shown in Figure 2, this is not the case. The consequence of the age distribution of the portfolio is that the number of properties requiring external painting annually will vary. The transformation of the portfolio from houses clad in high maintenance items such as timber/sheet/plank type products to low maintenance products such as brick/block will reduce this maintenance burden to a large extent. In this sample analysed, 43% of the properties can be classified in the high maintenance cladding group and the balance (57%) falling in the brick/block clad properties. Further analysis assessing the spatial distribution of the construction materials confirmed a higher proportion of properties with high maintenance cladding types (those types that require extensive external painting) are located in the older regions. A breakdown of the age distribution of the roofing material is displayed in Table 2. This table reveals that both colorbond sheeting and concrete tiles have been the predominant roofing materials in the last 20 years, while previous to this galvanized sheeting was the main roofing material used. A significant proportion (>50%) of properties over 50 years old have colorbond roofing and since this product was not available at the time of construction we can infer that these properties have had a roof replacement.

**Table 2 Age distribution of Roof Material**

| | Roof Material | | | | | | | Grand Total |
|---|---|---|---|---|---|---|---|---|
| Age Group | AC Sheet | Alum | Colorbond | Painted Gal sheet | Unpainted Gal sheet | Concrete Tile | Terracotta Tile | |
| 5 | 0.02% | | 2.57% | 0.05% | 0.06% | 1.29% | 0.12% | **4.11%** |
| 10 | | | **8.10%** | 0.08% | 0.23% | **9.92%** | 0.29% | **18.62%** |
| 15 | 0.03% | 0.03% | **8.87%** | 0.15% | 1.08% | **7.21%** | 0.08% | **17.45%** |
| 20 | 0.02% | 0.03% | 1.52% | 0.22% | 3.59% | 3.31% | 0.09% | **8.78%** |
| 25 | | 0.03% | 0.26% | 0.66% | **7.87%** | 2.88% | 0.14% | **11.84%** |
| 30 | 0.08% | 0.02% | 0.25% | 1.83% | **6.28%** | **4.24%** | 0.15% | **12.84%** |
| 35 | 0.12% | 0.02% | 0.35% | 1.48% | 0.92% | 1.69% | 0.40% | **4.99%** |
| 40 | 0.52% | | 0.17% | 1.71% | 0.59% | 2.62% | 0.86% | **6.47%** |
| 45 | 0.03% | 0.02% | 0.54% | 1.94% | 0.69% | 0.74% | 0.32% | **4.28%** |
| 50 | 0.69% | 1.02% | **4.30%** | 1.49% | 0.68% | 0.54% | 0.06% | **8.78%** |
| 55 | | | 1.26% | 0.15% | 0.09% | 0.11% | 0.05% | 1.66% |
| 60 | | | 0.05% | 0.02% | 0.05% | | | 0.11% |
| 70 | | | | 0.02% | 0.03% | 0.02% | | 0.06% |
| Grand Total | 1.51% | 1.16% | **28.25%** | **9.80%** | **22.16%** | **34.56%** | 2.57% | **100.00%** |

## 5 REGIONAL DIFFERENCES

To determine if there was any spatial variation in the maintenance preliminary assessment of the spatial distribution of the construction materials was completed. The proportion of cladding types from this sample varied throughout the regions with more than 70% of the properties in Southwestern Queensland, Central Queensland and Northwest Queensland clad in high maintenance cladding types. Similarly more than 80% of the properties in Chermside, Fortitude Valley and Stones Corner regions fall into this category. More than 50% of these properties are clad in timber. Conversely 80% of the properties in Woodridge, Redcliffe and Sunshine Coast regions are clad in brick veneer and more than 90% of the properties in the sample from the Gold Coast are clad in brick veneer. Both Stones Corner and the Sunshine Coast had very few observations in the sample (a total of 13) and as such their results should be interpreted carefully.

Analysis of the maintenance costs revealed the average repair cost of a property varied dependent on its location. This variation is due in part to higher costs of repair for components due to regional price differences. Regions including South-west Queensland and North-west Queensland recorded the highest average total maintenance cost with Fortitude Valley, Chermside and Redcliffe next although this cost was significantly lower. Mackay, Central Queensland, North Queensland and Far North Queensland had similar average maintenance cost to that of Redcliffe while the other regions all had a significantly lower average maintenance cost. Further investigation determined that the number of repairs per property varied significantly between the regions and this has impacted on the total maintenance costs for the dwellings. Table 3 below displays the average number of repairs per property for each region and the average repair cost index, which was derived by dividing the average repair cost for the region by the overall average repair cost. This statistic was supplemented with the regional cost index to provide an indication on the expected differences in the repair costs. The regional cost index provides an indication of the expected cost difference for a region compared to the base cost.

**Table 3 Repairs to properties and costs with comparative regional index**

| Region | Average number of Repairs per Property | Average Repair Cost Index | Regional Cost Index |
|---|---|---|---|
| *Sunshine Coast* | *2.29* | *0.74* | 1.00 |
| Ipswich | 3.91 | 0.86 | 1.00 |
| Capalaba | 3.25 | 0.90 | 1.00 |
| Inala | 2.77 | 0.93 | 1.00 |
| Chermside | 6.36 | 0.97 | 1.00 |
| Gold coast | 2.70 | 1.00 | 1.00 |
| Fortitude Valley | 4.81 | 1.03 | 1.00 |
| *Stones Corner* | *1.57* | *1.13* | 1.00 |
| Wide Bay-Burnett | 1.72 | 1.13 | 1.05 |
| Redcliffe | 1.98 | 1.19 | 1.00 |
| Mackay | 3.15 | 1.23 | 1.10 |
| Woodridge | 1.95 | 1.25 | 1.00 |
| Central Queensland | 3.30 | 1.31 | 1.05 - 1.30 |
| South west Queensland | 7.51 | 1.33 | 1.20 - 1.30 |
| North Queensland | 2.79 | 1.52 | 1.08 - 1.2 |
| Far north Queensland | 2.58 | 1.53 | 1.15 - 1.60 |
| North west Queensland | 2.63 | 1.58 | 1.50 - 1.90 |

Further analysis had to be carried out to determine if other factors such as construction material, age or some other unknown variable were confounding the results. It was expected that a combination of these factors would impact on the maintenance costs. The confounding factor expected to have the most effect and therefore the starting point for further analysis was the property age.

## 6 AGE DIFFERENCES

Figure 2 highlighted the transition of the portfolio dependent on the cladding type. It is apparent that those properties clad in brick veneer or concrete block will be relatively new properties while those properties clad in timber or one of the various sheeting or plank types are generally older. In all, 50% of the properties in the sample are less than 25 years old and only 13% of properties under twenty years old are constructed of high maintenance cladding – most being 15 to 20 year old properties clad in Hardie-Plank®.

Many of the houses in the southern Brisbane region are less than thirty years old; moreover the mean year of construction for the region is 1978. For the northern Brisbane region, the mean year of construction is 1968 and for the Sunshine Coast and Wide Bay regions the mean year of construction is 1984. The country regions mean construction year ranges from 1973 in Western Queensland (both Southwest Queensland and Northwest Queensland) to 1981 in North Queensland (Far north Queensland and North Queensland) with the Marlborough region (Central Queensland and Mackay) averaging 1978. If the average maintenance costs are influenced by the property age and not by the region as the above results indicated the costs for southern Brisbane and the Marlborough region should be similar as should the average cost for northern Brisbane and Western Queensland. We can see in Table 3 that the average number of repairs for Chermside, Fortitude Valley and South-west Queensland are all relatively high and Ipswich, Capalaba, Mackay and Central Queensland are also relatively similar indicating age may play an important role in predicting maintenance needs.

To complete this analysis the properties were classified into age groups. All properties aged five or less were classified into the property age group 5, those properties between six and ten years old (inclusive) were classified into property age group 10 and the balance were assigned according to their category following this principle. The analysis confirmed that property age is a significant factor to be considered when projecting a property maintenance requirement.

Table 1 below displays the results of the comparative analysis of the properties categorised by their age. The column, Percentage of sample, specifies the percentage of properties in the sample that each category contains. There are very few properties over 55 years old in the sample and as such the results for the properties in the 60 and 70 categories may not reflect the true statistics. The average repair cost index displays the results of the analysis and illustrates the effect of the properties age on the average repair cost. The index rating for the overall average is one while the index for the properties under fifteen years of age is less than 0.6. Conversely those properties, which are over forty years old, are more than 1.5 times the expected average repair cost. The repairs count column reveals the increase in the number of repairs identified for the properties as they age.

**Table 1 Repair count and average repair cost index categorised by property age.**

| Property age group | Percentage of sample | Average repair cost index | Repair count |
|---|---|---|---|
| 5 | 4.24% | 0.14 | 0.53 |
| 10 | 18.41% | 0.42 | 1.40 |
| 15 | 17.80% | 0.64 | 2.15 |
| 20 | 9.15% | 1.10 | 3.70 |
| 25 | 11.77% | 1.18 | 4.28 |
| 30 | 12.73% | 1.15 | 4.18 |
| 35 | 4.93% | 1.56 | 5.34 |
| 40 | 6.36% | 1.08 | 3.93 |
| 45 | 4.18% | 1.68 | 5.28 |
| 50 | 8.65% | 1.92 | 6.74 |
| 55 | 1.62% | 1.69 | 6.80 |
| 60 | 0.10% | 1.95 | 4.86 |
| 70 | 0.06% | 6.17 | 11.75 |

The values in Table 1 confirm an expected relationship between the average repair cost index and the repair count. Further investigation revealed a strong relationship between the two suggesting the major contributor for the maintenance costs are the number of repairs which is a linear function of age. A linear regression analysis was performed on the results, which suggests the average repair count is linearly related to the average repair cost index. Figure 3 below illustrates the output from the analysis and as can be seen the results are strong. The intercept (-0.03) was not statistically significant although the coefficient was, with the true coefficient value falling in the range of 3.09 to 4.03.

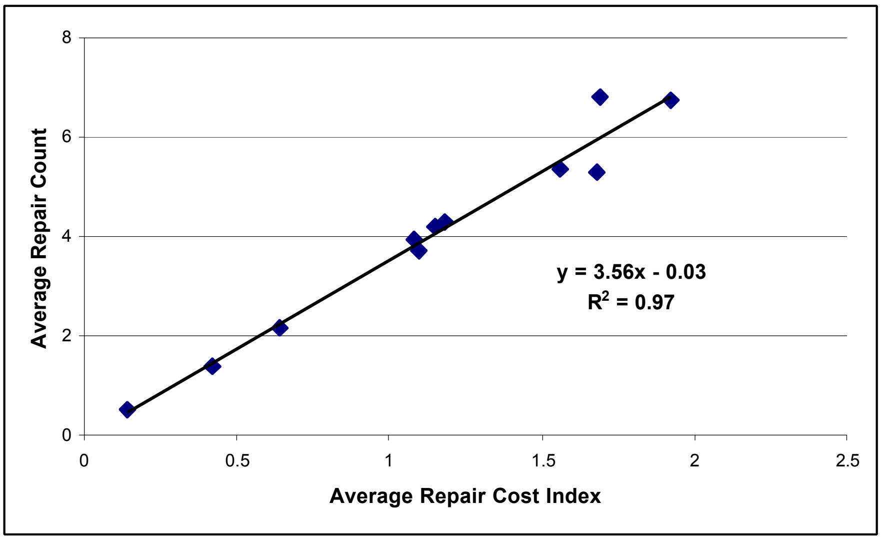


**Figure 3 Relationship between repairs count and maintenance costs**

Subsequent to this a regression, analysis was performed testing the relationship between the expected repair count and property age. The results indicate that the expected repair count increases by approximately 50% each five years. Given the results of the property age analysis and that the age distributions for the properties in the various regions differ, the regional analysis was reassessed and the effect of the region was determined after extracting the age effect. The total maintenance cost was divided by the number of repairs for each property to obtain the average repair cost. The results indicated that while there were differences in the mean maintenance costs for the regions dependent on the age of the properties, once standardised for age and regional cost differences; only South-western Queensland had consistently higher total maintenance costs. Additional analysis of this region found that the properties in this region had the highest number of repairs per property of the sample causing the highest average total maintenance cost. These properties include elements not generally found in other regions such as water tanks and stands, which may account for the increased number of repairs.

Table 3 shows the relationship between the average repair cost index for the regions and the regional cost index. This table confirms the regional differences are minimal and has little impact on the property maintenance needs. Only Redcliffe, Mackay, Woodridge and North Queensland had significant differences between the regional cost index and the average repair cost index and two of these regions have a large number of older properties. The different regions do however impact on the number of elements of a property and this can influence the total maintenance costs.

Estimating the survival probability

Given the findings from the analysis the focus turned to identifying a parametric distribution suitable to estimate the probability of survival for a property using the property age as a predictor. To help determine the appropriate decay function a simulation model was developed using Microsoft Excel. The simulation developed included several models and allowed the results to be compared and then the most appropriate model selected. The simulation models implemented used a range of likely parametric distributions, which the user could alter. Also the parameters of the functions and the expected life of components or the whole property could be altered by the user. The distributions available were:

- Exponential
- Gamma
- Linear
- Normal
- Poisson
- Weibull

The simulation was developed in the following manner. Firstly, the components were listed in a spreadsheet together with the expected life and their weighting relevant to the property, which was determined as a percent of total cost for the property. Then, the probability of failure for the component for each year of its expected life was calculated based on the distributions mentioned. For this simulation, as a component reached its expected life it was renewed. This process was repeated for each year of the expected life of the property, nominally 60 years. The simulation used a deterministic approach and therefore did not allow for any deviation from the expected life cycle of the components.

A stochastic model could be developed by including a random function that randomises the expected life of a component using the nominated expected life as a basis. Tucker and Rahilly (1990) used this approach in determining the lifecycle costs for housing assets. Since we are trying to model a theoretical outcome, it was felt that this approach might complicate the simulation unduly so it was decided this feature would not be included. Also, the likelihood of repairs not being carried out due to budget shortfalls was excluded, as this would require prioritisation of repairs and is beyond the scope of this analysis. The probabilities of failure for the components were then aggregated to property level to ascertain the likely state of disrepair of a property given its age.

Using the year range and probability of a property in the stock being built in a specific year information from the sample data, 40,000 properties were randomly generated. All of the properties met the minimum standards criteria for Housing Queensland properties. In this simulation, as a property reached its expected life it was replaced with a new property. All of the previously listed distributions were tested and the results compared. The Gamma distribution proved to be the parametric distribution the matched the empirical distribution developed from the sample.

The result of the simulation for the total stock is that the state of repair declines as the average age of the stock increases at property level as was the case for the sample analysed. The average age of the stock for this simulation in 2000 was 22 years and increased to an average age of 33 in 2019 implying that the maintenance needs in 2019 could be in the area of 40% greater per property. When the expected life of the properties was altered, either longer or shorter there were only minor changes to the outcome. As already noted, all distributions were tested with the Weibull and Gamma producing the best results for our purpose.

The Gamma distribution was chosen as it fit empirically and was theoretically sound. It is often used when a system comprises of components that have a specified expected life and the accumulation of a number of ‘sub-failures’ determine the failure of the system (Bain, 1978). There are two parameters for this distribution, the scale a and the shape ß parameters. Once the survival function was identified models were developed to supplement the curves with expert knowledge to ascertain the parameter estimates. This was done by developing models using Bayesian techniques and executing the model in an application known as WinBUGS. Essentially this is a form of sensitivity testing where the parameters of the functions are adjusted based on expert knowledge in the form of ‘informative’ priors. This is combined with the data and using MCMC metroplois algorthm the data is sampled and simulated. For this project between 1000 and 5000 iterations were completed depending on convergence and parameter estimates were extracted.

This model was developed to estimate the expected survival of a property and did not attempt to model the variability in a selection of properties as such use of the algorithm should be limited to a selection of properties. Testing confirmed the algorithm was reliable for a selection of dwelling but the reliability of the prediction reduces as the group size reduces. The smallest selection for reliable results is 30.

## 7 CONCLUSIONS

Both age and region were the two major factors that impacted on the expected repair costs for the dwellings. The region can be adequately accounted for by the regional cost index. The cladding type had little effect once the age and area factors were removed and need only to be considered when planning external painting projects. The roofing type effect is also negated once the repair costs are adjusted for the other factors. The statistical analysis conducted only considered differences in total cost of repair using expected costs as assessed by the inspectors.

The single factor that has the greatest influence on the cost of repairs of a dwelling was determined to be the age factor. The higher repair costs for older dwellings is due to the number of repairs needed which increases with age. The mean maintenance costs of the properties increased as the properties aged. This confirms the concerns of the Queensland DOH that older properties generally are in need of substantial repairs and those properties not scheduled for maintenance in the next three years will not be far off. From this analysis we were able to identify that the gamma distribution proved a suitable distribution to estimate the survival probability for a dwelling dependent on the property age. The property condition index was then supplemented using this information to provide a more accurate indicator of the property condition.

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# Durability Design For New Zealand Concrete Infrastructure

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**Summary: In mid 2000 Opus began a long-term research project to identify ways in which concrete materials and processes can be specified so that owners of concrete infrastructure in New Zealand can build and maintain their concrete structures more effectively. This paper describes investigations and initial findings from the project to date. A review of international trends in durability design indicated that performance based specification is seen as the solution to many of the shortcomings of prescriptive specification, but that its utilisation is hampered by difficulties in defining service life and in allowing for variabilities in materials and construction. A survey of New Zealand concrete designers and specifiers found that they too preferred a more performance based approach to specification. A survey of New Zealand infrastructure owners found that minimising construction costs is not as important to them as minimising whole of life costs, opening the way for more effective and enforcable specifications. A group of New Zealand concrete suppliers, researchers and consultants is currently formulating practical guidelines to performance specification for concrete construction.**

**Keywords: Asset management, concrete durability, durability design**

## 1 INTRODUCTION

In New Zealand, durability design for reinforced concrete structures is currently based on a prescriptive approach whereby parameters such as concrete cover depths and compressive strength are specified. The concrete structures design standard, NZS 3101 (Standards New Zealand,1995), gives limiting values for these parameters in various exposure conditions. These values are assumed to provide a 50-year service life, and are based on historical experience rather than any predictive tool, so the approach presents problems when new materials and different service lives are being considered.

In mid 2000 Opus began a long-term project that aims to identify ways in which concrete materials and processes can be specified so that New Zealand asset owners can build and maintain their concrete structures more effectively.

The project began with a review of approaches to durability design, and the composition, value and likely deterioration mechanisms of New Zealand infrastructure. This review was used to initiate discussion with local suppliers, designers and asset owners to identify preferred approaches to durability design, and from there to establish the changes in standards and other regulatory controls that have to be made before the desired level of durability design can be put into widespread practice. Asset owners were also asked about their expectations of long-term concrete performance, the level of confidence they need from service life predictions, and the price they are prepared to pay to achieve a designed durability performance.

The ultimate goal, a performance based approach, will take time to develop, and the immediate priority is to ensure that the current approach delivers the expected durability. This is being done by examining the performance of existing structures.

As the project proceeds towards improving design practices it will utilise international experience to identify and develop tests, acceptance criteria and models for predicting service life that are practical and suitable for New Zealand infrastructure and its physical, economic and regulatory environment. This paper describes the planned research and presents findings to date.

## 2 NEW ZEALAND'S CONCRETE INFRASTRUCTURE

### 2.1 Composition

The assets owned by transport, electricity generation and distribution, telecommunications, water and wastewater industries and institutions all comprise significant numbers of concrete structures. These include buildings, bridges, retaining walls, pavements, wharves, utility poles, water retaining structures, pipes, and hydraulic structures and are owned by many different organisations. Many of these have changed ownership and internal structure in the last 10-15 years. Possibly as a consequence there are few records about the numbers and types of structure, so it is difficult to quantify the significance of concrete infrastructure to the country's economy. Nevertheless we do know from available records and consulting experience that:

- Road bridges total approximately 17,000 and it has been estimated (Bruce et al, 1999) that over 10,500 include concrete as a major superstructure component. Replacement value is estimated at NZ$6000million (Bruce and Freitag, 2000).
- There are 16 major cargo-handling ports on the New Zealand coastline. The major structural components are wharves. Reinforced and prestressed concrete is the predominant material in wharf superstructures. Total berth length, excluding marina berths, is around 27km. Around 83% of New Zealand's exports by value and 99% by volume were carried by sea in 1998.
- A significant number of utility poles are reinforced or prestressed concrete. It has been estimated that more than 10,000 new precast concrete poles are produced annually. Poles in urban regions are utilised by several different services. The value of utility poles is often much less than the services they support.
- Of the approximately 400 dams in the country, 69 are concrete and most have associated hydraulic structures made from concrete.
- The national stormwater pipe network is approximately 7000km long. Most pipes bigger than 225mm diameter are concrete. Concrete pipe is not widely used for water supply but there are over 600 water treatment plants in the country, most if not all of which are substantially concrete. The wastewater pipe network is over 17,000km long with over 200 treatment plants. Most pipes over 300mm are spun concrete.

### 2.2 Factors affecting infrastructure durability

The corrosion of steel reinforcement is probably the most significant form of concrete deterioration in New Zealand. Although New Zealand's structures are not subjected to deicing salts, the fact that about half of our bridges (Bruce et al, 1999) and much of our other infrastructure is on or near the coast means that the ingress of marine salt is a significant risk. Any type of reinforced concrete structure with a relatively thin cross section and exposed to moisture can be affected. The symptoms and significance of reinforcement corrosion are widely recognised so incidences are generally reported relatively early.

Other types of deterioration encountered in New Zealand are more specific to particular environments and structures. Some may go unobserved or unreported unless damage is severe because their significance is less well understood. Soft water attack and leaching have been reported on structures exposed to flowing water or condensation. Few cases of sulphate attack have been reported and they are generally related to geothermal environments or chimneys venting coal-burning furnaces. Acid attack is a recognised problem for floors in food processing plants and in structures handling sewage. Alkali aggregate reaction (AAR) can cause expansion and cracking in concrete exposed to moisture if it contains sufficient alkali and reactive volcanic aggregates that are available in certain regions, but damage is generally minor. Surfaces exposed to flowing water are prone to erosion, which is more severe when solid material is carried in the stream, and there is a risk of cavitation when flow rates are high. Abrasion affects floors and pavements. Minor freeze-thaw attack has been observed on wing walls and kerbs on bridges in inland areas of the North Island but its extent is otherwise not known. Little is known about problems affecting foundations because they are not routinely examined.

## 3 DURABILITY DESIGN – HOW WELL HAVE WE DONE AND HOW CAN WE DO BETTER?

### 3.1 Current practice in New Zealand

The current New Zealand Standard for the design of concrete structures, NZS3101, ( Standards New Zealand, 1995) is based on a specifed intended life of 50 years. It prescribes minimum covers, curing periods and compressive strengths for concrete to protect against reinforcement corrosion in various environments, minimum strengths for abrasion resistance, minimum strengths and air contents for freeze-thaw resistance, and maximum chloride and sulphate ion contents. Unspecified precautions are required to be taken to minimise the risk of AAR damage or to ensure durability for the chosen design life in environments other than those covered by the exposure classifications defined as A1, A2, B1, B2 or C. This approach works with commonly-used materials and construction techniques but can obstruct the introduction of new products and methods or different service lives because of the extra cost of demonstrating the suitability of a proposed design. It also relies on past experience and "engineering judgement" to establish specification values.

NZS 3101 is currently under review, which gives the opportunity to assess its success in achieving the required durability, and to consider more flexible methods of specifying. Research at Opus Central Laboratories is investigating the durability of existing wharves, representing high risk structures, to ascertain whether the NZS 3101 durability requirements for preventing reinforcement corrosion in this environment are adequate if adhered to, and how they can be improved. It is hoped that this work will identify key factors in determining durability so that they can be built into models for predicting the durability of concrete structures. Findings will be reported elsewhere.

### 3.2 Current international practice

Much effort is being spent internationally on improving methods for predicting service life, particularly with respect to predicting the effects of reinforcement corrosion. In general terms, the main shortcomings of most models relate to the definition of “end of life” (time to corrosion initiation is easier to predict than, for example, time to cracking, spalling or loss of capacity, but may be unecessarily conservative), and the ability to accommodate variability in concrete properties and exposure conditions. Until these issues can be resolved and accurate probabilistic models created, compromises are necessary. One such approach is to measure properties that contribute to durability and use them to demonstrate how, for example, new materials perform compared to those that conform to existing specifications. This has its limitations because the new concrete might perform better in some tests and worse in others. Another compromise is to use simplistic deterministic service life prediction models, which are more straightforward but cannot cope well with the variabilities that often lead to premature deterioration.

Performance-based concrete specifications have well-recognised benefits but few organisations have adopted them because of the difficulty of establishing acceptance criteria or guidelines for the large number of properties that would need to be measured for a generic concrete standard. Individual owners or developers of substantial concrete assets may be able to afford the investment to establish technically sound specifications, acceptance tests and criteria suitable for their specific purposes. For example the Port Authority of New York and New Jersey specifies, as appropriate for any particular job, compressive and flexural strengths, permeability, bond strength, w/c, air content and chloride content (Bognacki et al, 2000). It has defined test methods and acceptance criteria for these properties, and to reinforce its commitment to producing durable concrete it enforces its specifications by determining contract payments according to conformance to the specified concrete properties.

### 3.3 What changes do New Zealand’s concrete designers want?

Following a review of international trends in durability design (Bruce and Freitag, 2000), representatives of the concrete design industry in New Zealand (NZ) were surveyed to find out how they believed standards needed to develop to improve durability design. The questions asked were based on those put to a UK group earlier in the year (BRE, 2000). The UK responses and a summary of the 18 NZ responses received are as follows.

*1. Should we define service life as an arbitrary number of years, or by criteria that address the functions that the structure is required to perform?*
UK: Service life needs to be more closely related to function, not an arbitrary number of years.
NZ: A combination: we need to have an acceptable probability of meeting appropriate performance criteria for a designed service life.

*2. Should we consider quality of people and processes as well as materials?*
UK: All three are essential. More needs to be done to enable people to deliver good products through good processes.
NZ: Yes. This may be improved through better training, supervision and testing practices or by accommodating them in safety factors as part of the design process.

*3. Do we raise acceptance criteria, introduce safety factors or introduce risk analysis to accommodate variable workmanship?*
UK: Safety factors are not appropriate, and risks arising from low quality need to be assessed.
NZ: Risk analysis was generally favoured. However we do not yet have enough data about the effects of various factors on durability to adopt effective risk analysis in durability design. Acceptance criteria also need to be addressed. They must be appropriate for the specific project.

*4. Should design allow for exposure of individual surfaces to wetting/drying?*
UK: Effects of water, wind driven water, dissolved salts and concrete water content need to be addressed.
NZ: Yes, but use worst case scenario or risk analysis to avoid over-complicating the process.

*5. Should different structures be protected by different methods or is it one standard for all structures?*
UK: No answer reported.
NZ: Question was ambiguous but answers indicated a desire for a single standard that allows different solutions for different situations.

*6. Do we need an integrated approach to design, construction and operation with the flexibility to allow for the needs of individual strucures?*
UK: Yes, we need to accommodate circumstances, needs and consequences of failure for individual applications.
NZ: Yes. “Owner’s manuals” for structures could be useful but can’t enforce owner involvement, and capabilities of owners and consultants will vary.

*Summary of designers’ survey results*
Overall the responses point to a desire for structure-specific durability design that accounts for variabilities in construction. New Zealand responses were similar to those from the UK workshop, so we should keep in touch with and utilise UK developments.

### 3.4 What do infrastructure owners want?

Concrete producers and designers are familiar with the issues of durability design, but does their approach address the needs and expectations of the infrastructure owners? Owners of structures associated with transportation, water supply, waste water treatment, institutions, ports, and electricity production and distribution were surveyed to find out what they understood by the concepts involved in durability design and whether their concerns were the same as those of the concrete industry.

*Sample*

A questionnaire was sent to 46 representatives of infrastructure owners. Organisations representing a range of types of infrastructure were selected. Willing and able individual participants within each organisation were identified by telephone before distributing the survey. At the time of writing, 42 individual responses representing the sectors indicated in Table 1 had been received from 29 organisations.

**Table 1. Infrastructure sectors represented by survey responses**

| *Sector* | *Roading* | *Water supply* | *Waste water* | *Electricity supply* | *Electricity production* | *Port* | *Airport* | *Rail* | *Institution* |
|---|---|---|---|---|---|---|---|---|---|
| *No. of respondents* | 12 | 3 | 9 | 4 | 4 | 3 | 2 | 1 | 4 |
| *% of respondents* | 28.6 | 7.1 | 21.4 | 9.5 | 9.5 | 7.1 | 4.8 | 2.4 | 9.5 |

*Questions and responses*

The survey comprised eight groups of statements, each around a particular theme. Owners were asked to indicate whether they "strongly agreed", "mostly agreed", were "uncertain", "mostly disagreed" or "strongly disagreed" with each statement. The statements are listed in tables 2-8, along with the distribution of responses to each statement expressed as a percentage of all responses to that statement. Some key features of the general distribution of responses to each theme are described below. Responses were not analysed for differences between all the sectors because the sample size was too small and its makeup not equally representative of all sectors. Thus the comments refer to the overall distribution of responses.

However there were enough responses representing the roading and waste water sectors to look at these in more detail. The level of agreement with each statement was coded from 1(strongly agree) to 5 (strongly disagree) and the "mean response" for each statement calculated for the the roading sector and for the waste water sector. The means from these individual sectors were compared with the means from all sectors other than the individual sector, using a t-test to distinguish significant differences between the means at the 0.05 probability level. Comments are made below on the significant differences that were revealed.

Inspection and maintenance (table 2). Although 90% of responses indicated that structures are inspected after potentially-damaging events, 29% indicated having no formal programme of regular inspection. 49% claimed to rely on site personnel reporting damage during their normal activities, although this conflicts with the 67% who claimed to have a formal inspection programme. 80% reported recording observations and 71% claimed to have a systematic reporting system. 88% actively maintain their concrete structures, 71% having a formal maintenance programme. Overall this suggests that the owners manage their structures proactively, indicating that they are aware of the risks of concrete deterioration and seek to control it to avoid severe damage and possible structural failure.

**Table 2. Inspection and maintenance**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| *THEME: INSPECTION AND MAINTENANCE* What is your organisation's present approach to inspecting and maintaining concrete structures? "Inspection" means specifically visiting a structure to observe and record its condition. "Maintenance" includes activities that keep the structure in good working order (eg cleaning or painting), as opposed to repair, which is rehabilitating a damaged structure. | | | | | |
| Concrete structures are inspected at intervals prescribed by a formal inspection programme. | 45.2 | 21.4 | 4.8 | 19.0 | 9.5 |
| Concrete structures are inspected after events that might cause damage (immediate or delayed). | 53.7 | 36.6 | 0.0 | 4.9 | 4.9 |
| Concrete structures are not formally inspected. We rely on site personnel reporting any need for maintenance or repair observed during their normal activities. | 12.2 | 36.6 | 0.0 | 22.2 | 29.3 |
| The condition of concrete structures is recorded for future reference. | 38.1 | 42.9 | 4.8 | 11.9 | 2.4 |
| We keep systematic records of observations made at each inspection (eg standard report forms, database). | 38.1 | 33.3 | 2.4 | 23.8 | 2.4 |
| Concrete structures are maintained according to a formal programme (this includes "according to need as indicated by inspection"). | 23.8 | 47.6 | 2.4 | 16.7 | 9.5 |
| Maintenance is carried out only when convenient. | 4.9 | 29.3 | 4.9 | 34.1 | 26.8 |
| No maintenance is carried out on concrete structures. | 4.8 | 7.1 | 0.0 | 26.2 | 61.9 |

It was thought that the roading sector might have better-developed inspection and maintenance programmes than other asset owners because the number and accessibility of their structures means such activities are more likely to be carried out and because of the public safety issues involved. The national road controlling authority, Transit New Zealand, has a well-developed inspection and maintenance programme and provided six of the 12 responses. The equality of means analysis showed convincingly that the roading sector's programmes for routine inspection, documentation and maintenance were indeed more formal and systematic than of the other sectors. In contrast, there was no significant difference in the mean response to the statement about inspection after potentially damaging events.

The wastewater sector responses indicated a less formal approach to inspection than other sectors, relying more on observations of site personnel.

Design for maintenance (table 3). 80% of respondents felt they could develop an appropriate inspection and maintenance schedule when designing a new structure, and 70% believed they could commit to carrying it out. This suggests that durability design for infrastructure may be able to rely on infrastructure owners taking a more active interest right from the time of design in how a new structure will be managed, compared to design for structures such as commercial buildings which are prone to changes in use and ownership during their lifetime.

**Table 3. Design for maintenance**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| THEME: DESIGN FOR MAINTENANCE<br>If Standards and Building Code etc requirements allowed, could you, as the owner, help to optimise the durability and the construction costs of a new concrete structure by considering at the design stage how you are going to monitor and maintain it? | | | | | |
| I could develop an appropriate inspection and maintenance schedule in conjunction with the design of a new concrete structure to minimise long-term costs. | 32.5 | 47.5 | 10.0 | 10.0 | 0.0 |
| I could commit to carrying out the above schedule for the foreseeable future of the structure. | 32.5 | 37.5 | 20.0 | 10.0 | 0.0 |

The roading sector ‘agreed” a little (but not significantly) more strongly that they could develop such schedules, and agreed significantly more strongly that they could commit to them. This reflects their well-developed existing commitment to inspection and maintenance.

Need for repair during service life (table 4). Although 56% of respondents indicated that there should be no need for repair, 90% expected that there would be minor or localised deterioration that would need repair during a structure’s lifetime. 24% expected some significant repair while 64% did not expect that significant repair would be needed and 12% were unsure. Thus despite some inconsistency in the responses some risk of deterioration is acceptable to most owners – the level is likely to depend on the individual structure.

**Table 4. Expectation of need for repair during service life**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| *THEME: REPAIR*<br>What level of repair do you expect a concrete structure to need during its service life? | | | | | |
| Within a concrete structure’s service life there should be no need to repair deteriorating concrete. | 4.8 | 52.4 | 2.4 | 33.3 | 7.1 |
| Within a concrete structure’s service life there will be some minor or localised concrete deterioration that needs repair. | 38.1 | 52.4 | 4.8 | 2.4 | 2.4 |
| Within a concrete structure’s service life one or more cycles of significant repair to deteriorated concrete will be needed. | 0.0 | 23.8 | 11.9 | 31.0 | 33.3 |

The wastewater sector had a significantly stronger expectation of no repair than the other sectors. This could reflect the inconvenience of having to divert flows when repairing wastewater structures. It could also be interpreted as meaning they replace rather than repair, although they showed no stronger preference to replace than other sectors (see “what happens at end of service life” below).

Service life (table 5). Service life had both financial and functional significance to most respondents. 74% considered that it was determined by concrete durability. This was felt significantly more strongly by the wastewater sector. 88% believe that they should be able to extend the service life past the original design value as a structure ages. Most believed that the design life of a structure, or parts of a structure, should account for individual circumstances (eg local variations in exposure to moisture or aggressive agents).

**Table 5. Service life**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| THEME: SERVICE LIFE<br>What does "service life" mean to you? The following statements seek to find out how important a predetermined "service life" is to you, and whether it is based on expectations of functionality or concrete durability. | | | | | |
| I need to assume my concrete structures will last for a specified service life for financial management purposes. | 50.0 | 40.5 | 0.0 | 9.5 | 0.0 |
| I need my concrete structures to last for a specified service life for engineering/functional purposes. | 66.7 | 31.0 | 0.0 | 2.4 | 0.0 |
| The service life of my concrete structures is determined by the durability of the concrete. | 16.7 | 57.1 | 11.9 | 9.5 | 4.8 |
| As a concrete structure gets older I expect to be able to extend its service life past the original designed minimum. | 21.4 | 66.7 | 9.5 | 2.4 | 0.0 |
| All concrete structures of the same type should have the same designed service life. | 7.3 | 43.9 | 17.1 | 24.4 | 7.3 |
| All parts of a structure should have the same service life (eg guardrails, which are relatively easy to replace, and substructure of a bridge). | 0.0 | 22.2 | 12.2 | 56.1 | 9.8 |
| The designed service life of a structure should take into account its individual circumstances (eg exposure, accessibility). | 47.6 | 47.6 | 2.4 | 2.4 | 0.0 |

What happens at the end of service life (table 6). Although more respondents preferred to upgrade rather than replace structures, preferences were spread over the categories with a large proportion "unsure". Again, this probably reflects the need to consider the individual circumstances of different structures.

**Table 6. What happens at the end of service life**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| *THEME: WHAT HAPPENS AT THE END OF SERVICE LIFE?*<br>The following statements ask what you prefer to do when a concrete structure no longer meets critical performance requirements, ie its design and/or condition are no longer adequate (eg structural capacity for a rural bridge, aesthetics for a prominent structure in public view). "Upgrade" means modifying and/or repairing the existing structure. | | | | | |
| I prefer to upgrade an existing concrete structure at the end of its service life. | 17.1 | 43.9 | 22.0 | 14.6 | 2.4 |
| I prefer to replace a concrete structure at the end of its service life. | 12.2 | 22.2 | 29.3 | 31.7 | 4.9 |
| *THEME: REPLACEMENT OF CONCRETE STRUCTURES*<br>In the design of a new concrete structure, what do you anticipate will be the main reason that it would ultimately be replaced? | | | | | |
| Replace because its design (eg capacity, location) is no longer appropriate and upgrading is not practical. | 28.2 | 38.5 | 17.9 | 15.4 | 0.0 |
| Replace because it has deteriorated to such an extent that ongoing repair is not economic or practical. | 23.7 | 31.6 | 18.4 | 26.3 | 0.0 |
| No single main reason to replace - both design and condition are no longer adequate. | 21.1 | 42.1 | 13.2 | 23.7 | 0.0 |

Construction costs vs whole of life costs (table 7). There was an overwhelming preference to minimising whole of life costs rather than minimising construction costs. This was somewhat surprising given the apparently cost-driven approach to many construction and repair contracts. It indicates that infrastructure owners take a long-term view to asset management even though their corporate procurement policies may not always enable whole of life benefits to be assessed when comparing tenders. This might not be the case for developers of commercial property. Enthusiasm for paying more for ongoing preventive maintenance was not quite as strong, favoured by 63% of respondents and not favoured by 24%.

The wastewater sector disagreed even more strongly than the other sectors with the concept that maintenance and repair costs are less important than minimising construction costs.

**Table 7. Construction costs vs whole of life costs**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| THEME: WHOLE OF LIFE COSTING<br>In the design of a new concrete structure, what do you consider to be the relative importance of construction costs, maintenance and repair costs and whole of life costs? | | | | | |
| The cost of subsequent maintenance and repair is not as important as minimising construction cost. | 0.0 | 9.8 | 2.4 | 53.7 | 34.1 |
| I need to minimise whole-of life costs. | 54.8 | 40.5 | 2.4 | 2.4 | 0.0 |
| I am prepared to pay higher construction costs to reduce maintenance and repair costs. | 31.7 | 58.5 | 7.3 | 2.4 | 0.0 |
| I am prepared to pay higher construction costs to avoid the disruption of maintenance and repair | 26.8 | 61.0 | 4.9 | 7.3 | 0.0 |
| I am prepared to pay more for ongoing preventive maintenance (eg maintaining a coating or a corrosion monitoring system) to reduce the risk of deterioration | 12.2 | 51.2 | 12.2 | 22.2 | 2.4 |

Allowance in design for future deterioration (table 8). 83% of respondents assumed that deteriorating concrete would be repaired before structural capacity is reduced and the same proportion indicated that they would not accept a risk of future strength reductions. 48% of respondents indicated that they preferred to allow for significant deterioration by designing for a higher load carrying capacity.

**Table 8. Allowance in design for future deterioration**

| *Theme/Statement* | *% of responses* | | | | |
|---|---|---|---|---|---|
| | *Strongly agree* | *Mostly agree* | *Unsure* | *Mostly disagree* | *Strongly disagree* |
| THEME: RISK<br>In the design of a new structure, how would you prefer to accommodate the risk of eventual strength reduction due to concrete deterioration? | | | | | |
| I would prefer to allow for significant concrete deterioration and require a higher load carrying capacity to be designed into the structure. | 4.8 | 38.1 | 9.5 | 40.5 | 7.1 |
| I would prefer to allow for significant concrete deterioration and accept a future structure strength lower than Building Code requirements because of the reduced service life. | 4.9 | 2.4 | 9.8 | 43.9 | 39.0 |
| I would assume that deteriorating concrete would be repaired before strength is reduced. | 22.0 | 61.0 | 9.8 | 7.3 | 0.0 |

*Summary of owners' survey results*

Overall, the results of the survey indicate that the approaches being taken by infrastructure designers do reflect the interests of their clients. The results suggest that owners may be more willing than previously thought to contribute to the optimisation of durability design by developing maintenance programmes, accepting the need for damage to minor or isolated deterioration, and accepting higher construction costs to reduce the risk of future deterioration. Owners are keen to see the circumstances of individual structures taken into account in durability design. Beyond these generalisations, different owners and different types of structure will have different needs, for example deterioration is likely to be less acceptable in structures that must be decommissioned to repair, such as those in wastewater systems. Thus before committing to a design philosophy the designer and client must both understand the likely effects of different operating, management and maintenance programmes on the potential durability of a structure.

### 3.5 How do we start to change our approach?

In mid-2001 the Cement and Concrete Association of New Zealand (CCANZ) established several industry focus groups to address issues of concern to the concrete industry. One of these was tasked with establishing a methodology for performance-based specification. Its aim is to develop simple specification guidelines for the construction of floors (the main use of concrete in New Zealand) and marine structures (the structure most at risk from significant durability problems) based on overseas and local experience. Recognising that there is a lack of tests that are well-enough developed to use for acceptance, these guidelines will describe the factors that can and can't currently be controlled by performance-based specification and will give guidance on appropriate means of specification. It has been suggested that for many situations a prescriptive approach will be most appropriate. The focus group's progress will be reported at the conference.

## 4 CONCLUSIONS

Although difficult to quantify because of the diversity of structures, ownership and management practices, concrete structures comprise a significant part of New Zealand's infrastructure. Different structures are prone to different types of deterioration but the most significant is the corrosion of steel reinforcement because it can affect any reinforced concrete structure.

Designers are keen to see a more flexible approach to durability design, taking into account the circumstances of individual structures, and using risk analysis to account for variability.

Contrary to the impression given by common procurement practice, infrastructure owners do take a long-term view to asset management and are as keen as designers to optimise whole of life costs. There is therefore a real possibility of improving the durability of concrete infrastructure by using - and enforcing - performance based specifications tailored for individual project needs, without being compromised by an overriding need to minimise construction costs. The key to achieving the desired durability will be to make sure the designer and client understand the consequences of the way in which a structure is built, operated and maintained.


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# A Study Of The Polarisation Techniques For Corrosion Rate Measurement In A Steel-Concrete System

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**Summary: This paper presents the results from a study of polarisation techniques for determining the corrosion rate of steel in reinforced concrete. Currently, two types of instruments for field measurement of corrosion rate of steel in reinforced concrete are commercially available: linear polarisation technique (e.g. GECOR from Spain) and transient technique (e.g. GalvaPulse from Denmark). The measured corrosion rates from these two different techniques differ very much. The corrosion rates measured by the transient polarisation technique are usually higher than those by the linear polarisation technique. The differences are sometimes larger than one order of magnitude. The experimental results from this study show that the calculated polarisation resistance is strongly dependent on the polarisation duration, no matter if the steel is corroded or not. The effect of polarisation current appears not significant if the responded potential shift is small enough to assure a linear polarisation. The results show that the differences in corrosion rate measured by the above two different polarisation techniques are mainly attributed to different polarisation durations.**

**Keywords: Concrete, corrosion, electrochemical measurement, polarisation, steel.**

## 1 INTRODUCTION

Corrosion of steel in reinforced concrete structures is a serious problem around the whole world. Every year a lot of money was spent for repairs of highway bridges due to the corrosion damage. In addition to the direct economic losses, the failure of expensive infrastructures can be tragedies and can have serious economic, environmental and social consequences. If corrosion of reinforcement would be detected at an early age, proper measures could be taken to prevent or delay the corrosion damage, thus great amount of labour and money could be saved in post-repair work, and potential serious accidents could be avoided. It is a common understanding that corrosion of steel in concrete is an electrochemical process involving anodic and cathodic reactions. This is why most of the non-destructive methods for measuring corrosion of steel in concrete are based on the electrochemical principles.

Different electrochemical methods are available for measuring corrosion of steel in concrete (Song & Shayan 1998). One of the widely used electrochemical method is the half-cell potential mapping, which has been standardised as ASTM C 876 since 1977. The significant advantage of the potential mapping technique is its simplicity and rapidity. Since many factors can influence the half-cell potential, it has been reported that there was no real correlation between half-cell potential and corrosion current density (Escalante 1990; Broomfield et al 1994), corrosion states (Cigna et al 1993) or polarisation resistance (Oshiro et al 1992). Therefore, the potential mapping technique can only be used for locating the possible corroding areas of a reinforced concrete structure (Elsener 1998).

Among different electrochemical methods, polarisation techniques are probably the most suitable methods for field measurement of corrosion rate of steel in reinforced concrete. There are two types of polarisation technique: linear polarisation and transient polarisation, the latter is very promising for field mapping due to its rapidity (a few seconds per measurement). The measurement results from these two different polarisation techniques are, in many cases, inconsistent (Elsener 1995; Tang et al 2001). The corrosion rates measured by the transient polarisation technique are usually higher than those by the linear polarisation technique, especially when steel is in a passive state (Elsener 1995). Although Polder et al (1994) concluded from their comparison tests that the galvanostatic pulse method (GPM) was shown to provide corrosion rates similar to classical Rp (linear polarisation) methods, the reported corrosion rates by the GPM were still as about 1.5-3 times large as those by the classical one. It has been noticed that, in Polder et al’s comparison tests, the difference in polarisation durations between those two methods was small, that is, the GPM had a polarisation duration of 10 seconds and the classical one 20-30 seconds in their experiments. Tang et al (2001) found in their comparison tests that the differences in corrosion rate measured by GalvaPulse and GECOR were sometimes larger than one order of magnitude when they used the polarisation duration of 5 seconds for GalvaPulse and 100 seconds for GECOR. With such a large difference, it is difficult for engineers to apply the measured corrosion rates in service life prediction. This study attempted to find out the reasons why the measured results from these two techniques are far different.

## 2 THEORETICAL BASES BEHIND POLARISATION TECHNIQUES

Stern & Geary (1957) first presented the relationship between polarisation resistance $R_p$ (often in kΩ) and corrosion current density $i_{corr}$ (often in μA/cm$^2$), which is expressed as:

$$i_{corr} = \frac{B}{AR_p} \tag{1}$$

where $B$ is a constant (often in mV) and $A$ is the polarised area (often in cm$^2$). Theoretically, $B$ is determined by cathodic and anodic Tafel slopes $\beta_c$ and $\beta_a$ (the slopes of cathodic and anodic polarisation curves in Tafel linear regions):

$$B = \frac{|\beta_a \cdot \beta_c|}{|\beta_a| + |\beta_c|} \tag{2}$$

Once $i_{corr}$ is known, the rate of corrosion in terms of section loss of steel can be easily derived from Faraday's law:

$$\frac{\Delta x}{\Delta t} = \frac{M}{zF} \cdot \frac{i_{corr}}{\rho} \tag{3}$$

where $x$ is the section loss of steel, $t$ is the corrosion duration, $M$ is the molecular weight of metal ($M$ = 56 g/mol for Fe), z is the number of ionic charges ($z$ = 2 for Fe), $F$ is the Faraday constant ($F$ = 96480 C/mol or A/s/mol) and ρ is the specific density of metal (ρ = 7.8 g/cm$^3$ for Fe). Thus, 1 μA/cm$^2$ of corrosion current density can easily be converted to 11.5 μm/yr of section loss, or 23 μm/yr of diameter reduction of steel bar.

It should be kept in mind that the above equations are valid only for homogeneous corrosion and their application to chloride induced localised corrosion is questionable. Some theoretical and experimental aspects of this point have been discussed elsewhere (e.g. Elsener, 1998b). Even assuming that these equations are applicable to chloride induced corrosion, each of the parameters ($B$, $A$ and $R_p$) in Equation (1) still involves complexities and difficulties. The first question is how to determine the actual value of $B$. Song (2000) analysed $B$ values on four different situations and concluded that $B$ value may vary from 8 mV to infinite. Conventionally, the value of $B$ for steel-concrete systems is regarded to be within the range between 25 to 52 mV (Andrade & González 1978; Alonso et al, 1988; Aarup & Klinghoffer 1996), and $B$ = 26 mV is usually assumed (Alonso et al, 1988; Rodríguez et al 1995; Elsener et al 1997). This implies that, if the actual value of $B$ varies from 13 to 52 mV, the error caused by the assumption of $B$ = 26 mV is somewhat a factor of 2.

The second question is to determine the actual polarised area $A$. Since the steel bars in concrete are often "infinite" long when compared with the length of a counter electrode, the polarised area is not necessarily equal to the corresponding length of the counter electrode. This problem has been, at least to some extend, solved by using "guard electrode" to confine the direction of polarisation current flowed through the counter electrode (Feliu et al 1990; Homma et al 1992).

The last but most important question is to determine the polarisation resistance. Different techniques, such as polarisation, impedance spectroscopy, noise analysis, etc., could be employed for determination of $R_p$. The discussions in this paper will be limited to polarisation techniques. For linear polarisation techniques, $R_p$ is simply expressed as:

$$R_p = \left.\frac{\partial E_p}{\partial I_p}\right|_{I_p \to 0,\, E_p \to E_{corr}} \approx \left.\frac{\Delta E_p}{I_p}\right|_{E_p \to E_{corr}} \tag{4}$$

where $E_p$ and $I_p$ are the polarisation potential and current, respectively, and $E_{corr}$ is the corrosion potential, which is measured using the half-cell technique. The linearity of $\partial E_p/\partial I_p$ is a fundamental assumption of the linear polarisation technique. This means that $I_p$ must be very small in order to keep $E_p \to E_{corr}$. Normally, $\Delta E_p$ should be less than 20 mV (max. 60 mV).

There are two ways to obtain $\Delta E_p/I_p$:

- Potentiostatic way – applying a constant external potential $\Delta E_a$ and measuring the response current $I_p$.
- Galvanostatic way – applying a constant external current $I_p$ and measuring the response potential $\Delta E_a$.

It should be pointed out that the applied or recorded $\Delta E_a$ is the total potential drop, which is a sum of $\Delta E_p$ and $\Delta E_\Omega$, the latter is called "ohmic drop" and attributed to the ohmic resistance $R_\Omega$ between the steel reinforcement and the counter electrode.

$$\Delta E_p = \Delta E_a - \Delta E_\Omega = \Delta E_a - I_p \cdot R_\Omega \qquad (5)$$

The actual value of $R_\Omega$ is dependent on many factors, such as the porosity of concrete, the compositions of pore solution, the moisture condition of concrete, the shape of counter electrode, etc. Especially when concrete is dry, $R_\Omega$ may be very large. Simply using $\Delta E_a$ instead of $\Delta E_p$ in Equation (4) may result in very large error (overestimated $R_p$ and, as a consequence, underestimated $i_{corr}$). Therefore, it is important to know the actual value of $R_\Omega$ in order to correctly quantify the polarisation resistance $R_p$.

According to the above two ways to obtain $\Delta E_p/I_p$ the linear polarisation technique is a kind of stationary measurement, that is, the measurement is under the assumption that the response current or potential is under a stationary state. Due to double layer capacitance in a steel-concrete system, it will take some time to reach a stable response current or potential, especially for the passive steel under a very low applied potential or current. According to the data reported by Videm & Myrdal (1997), a stationary state could not be reached even for a measurement period of 100 hours!

Not like the linear polarisation technique, which measures response signals under the stationary state, the transient polarisation technique, e.g. galvanostatic pulse technique, measures response signals under the non-stationary (transient) state. This technique has been studied in the laboratories by many researchers (Elsener & Böhni 1983; Ijsseling 1986; Newton & Sykes 1987; Rodríguez et al 1994; Elsener et al 1994; Gower et al 1996; Videm & Myrdal 1997), and been introduced to the steel-concrete system for in-site measurement since 1988 (Elsener 1988).

When applying a constant current to the system, under the non-stationary state the response of potential $\Delta E_a$ will change with time $t$. By assuming the steel-concrete system is like a Randles circuit, one can express the responded potential as

$$\Delta E_a = I_p R_p \left(1 - e^{-\frac{t}{R_p C_{dl}}}\right) + I_p R_\Omega \qquad (6)$$

where $C_{dl}$ is the double layer capacitance. Curve-fitting equation (6) to the measured values gives three parameters: $R_\Omega$, $R_p$ and $C_{dl}$.

One of the significant advantages of the transient polarisation technique is its rapidity (normally a few seconds per measurement), implying that one can map a large area of the structure within a short time. Therefore, it is a promising technique for in-site measurement of corrosion in concrete.

Another way to evaluating the measured data is to transform the above equation to a linear form:

$$\ln(\Delta E_{max} - \Delta E_a) = \ln(I_p R_p) - \frac{t}{R_p C_{dl}} \qquad (7)$$

where $\Delta E_{max}$ is the maximum potential reached after a certain long time ($t > 5 \times R_p C_{dl}$). Extrapolation of this straight line to t = 0, using least square linear regression, yields an intercept corresponding to $\ln(I_p R_p)$ with a slope of $1/(R_p C_{dl})$. Thus $R_p$ can be obtained from the intercept and $C_{dl}$ can then be obtained from the slope (Elsener et al, 1997).

The disadvantage in the latter approach is the estimation of $\Delta E_{max}$. When a steel bar in concrete is under passive corrosion conditions, the value of $R_p C_{dl}$ may be very large. It will take a long time to obtain a reasonable value of $\Delta E_{max}$. In this case, it loses the rapidity of the technique. In fact, when polarisation duration is long, equation (6) is reduced to equation (4), that is, there is no theoretical difference between transient and linear polarisation techniques. Therefore, the assumption of linearity should also be valid for the transient polarisation techniques.

## 3 EXPERIMENTAL INVESTIGATION

As mentioned above, both linear and transient polarisation techniques are, in fact, based on the same theory but different approaches. Therefore, the measurement results from these two techniques should not be far different. In order to find out the reasons why the measured results from these two techniques are far different, an experimental study was carried out to investigate the effects of polarisation current and duration on polarisation resistance.

## 4 EXPERIMENTAL

Reinforced concrete slabs of size 250×250×70 mm, with and without chlorides in the mixture, were cast in the laboratory. Two round carbon steel bars of diameter 10 mm were embedded in each slab. Before casting concrete, the end portions of bars were coated first with cement paste and then with epoxy to avoid unexpected crevice corrosion. The location of steel bars in a slab is illustrated in Fig. 1. A computerised 16 bits data acquisition system was employed to supply cathodic polarisation current to the steel bar and at the same time collect the responded potentials. The ratio of current density on the guard electrode to that on the counter electrode is defined as $\gamma$,

$$\gamma = \frac{I_{GE}/A_{GE}}{I_{CE}/A_{CE}} = \frac{i_{GE}}{i_{CE}} \qquad (8)$$

where the subscripts GE and CE represent guard electrodes and counter electrodes, respectively. If not otherwise stated, $\gamma = 1$ was used in the experiment. The measurement arrangement is shown in Fig. 2. The concrete slabs without chlorides were cured in closed plastic boxes with water as moist medium, while the slabs with chlorides were cured in closed plastic boxes with saturated KCl solution as moist medium. The measurement data reported in this paper were taken at an age of 22 weeks.

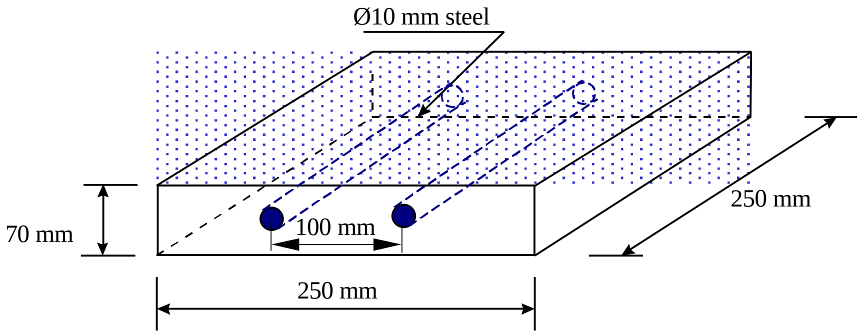



**Figure 1. Location of steel bars in a concrete slab.**

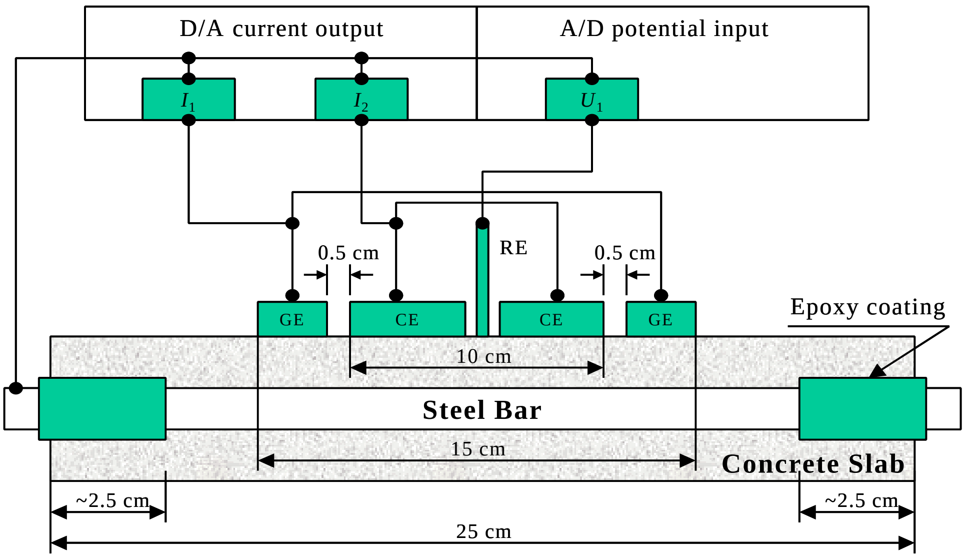
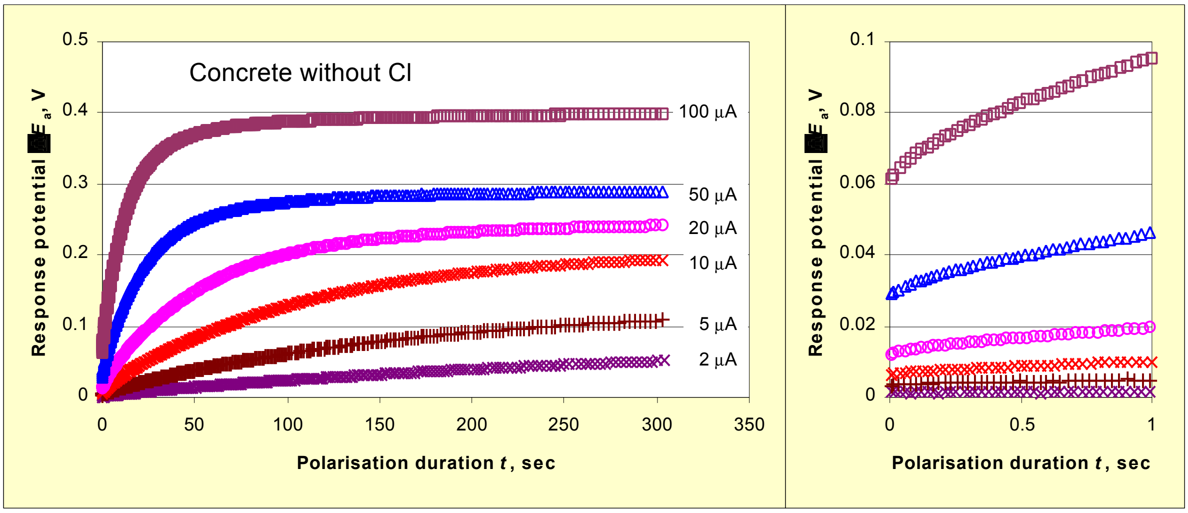



**Figure 2. Experimental arrangement for polarisation measurement. The effective polarised area of steel bar is assumed to be 33 $cm^2$ (p×D×L = 3.14×1×10.5).**

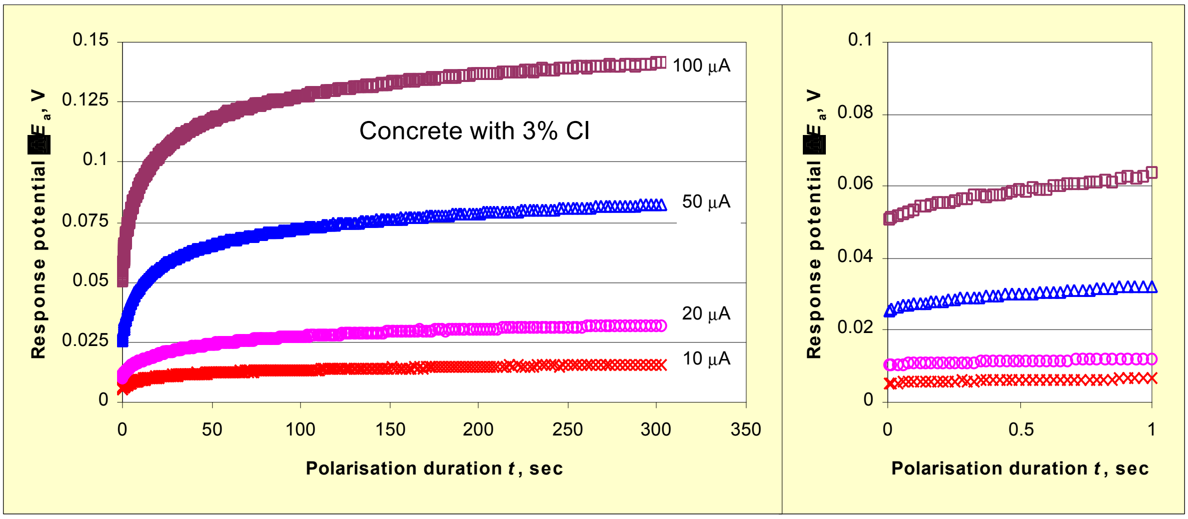
## 5 TEST RESULTS

The potential curves measured from two separate bars in each concrete slab are in general very similar. Owing to the page limitation, only those from one of the two bars are presented here, as shown in Figs. 3 and 4. As expected, the responded potentials from the concrete without chlorides are significantly higher than that from the concrete with added chlorides. The responded potential increases with the intensity of an imposed galvanostatic current, but the increment is non-linear, as will be discussed later. When a potential difference is less than 100 mV, e.g. the curves of the polarisation current less than 10 ? A in Fig. 3 and all the curves in Fig. 4, the stationary polarisation could not be achieved even after a polarisation duration of 300 seconds. This is in agreement with the findings reported by Videm & Myrdal (1997). When the imposed galvanostatic current is larger than 20 ? A for passive steel, the responded potential looks close to a stationary state after more than 100~200 seconds polarisation (see Fig. 3). However, the potential difference in all of these curves is over 200 mV, which is far beyond of any of the conditions of linearity. Therefore, the stationary state of these curves could be some artefacts.




**(a)** **(b)**

**Figure 3. Potential response from concrete without chlorides. (a) full data; (b) response in the first second.**




**(a)** **(b)**

**Figure 4. Potential response from concrete with 3% chloride by mass of cement. (a) full data; (b) response in the first second.**

The initial potential response (at $t$ = 0) reflects the ohmic resistance of concrete. It can be seen from Figs 3 (b) and 4 (b) that, at the same current level, the initial potential response from the concrete with 3% chlorides is not significantly lower than that from the concrete without chlorides, implying that the resistivity of concrete is not a decisive parameter to corrosion of steel. This should be true, because the resistivity of concrete is strongly dependent on the content of moisture and the concentration of ions, especially hydroxides, in the pore solution. In this study, the concrete without chlorides were cured under a moist condition (>95%RH), while the concrete with added chlorides under a condition of about 85%RH (saturated KCl), the former has a higher moisture content than the latter. Therefore, addition of chloride or corrosion of steel in concrete may not necessarily mean a significant increase in conductivity, or decrease in resistivity, of concrete.

Since a potential shift of 60 mV from the corrosion potential may be the maximal limitation to a condition of linearity, we only take those data of potential shift less than 60 mV as valid data. By curve-fitting equation (6) to these valid data from different polarisation durations, we will obtain different values of $R_\Omega$, $R_p$ and $C_{dl}$, as shown in Fig. 5. It is noticed that each fitted curve is in a good agreement with the corresponded data, implying that the polarisation behaviour of a steel-concrete system only "time-dependently" obeys the Randles circuit, and a better model is needed to describe this time-dependent behaviour. The increase in the curve-fitted $R_\Omega$, although not so significant when compared with those in $R_p$ and $C_{dl}$, is also due to the increased potential responses, which statistically reduced the weight of the initial points of a $\Delta E_a$-$t$ curve in the curve-fit.

Nevertheless, in this study equation (6) was applied to the valid data of different durations of each $\Delta E_a$-$t$ curve to obtain the relationships between polarisation resistance and polarisation duration at different intensities of polarisation current, as shown in Fig. 6. It is evident that the polarisation duration has remarkable effect on the curve-fitted polarisation resistance, while the effect of polarisation current seems not significant when compared with the effect of polarisation duration. The fluctuations in the curves of 2 and 5 ? A are probably due to the measurement uncertainty when the potential response was low.

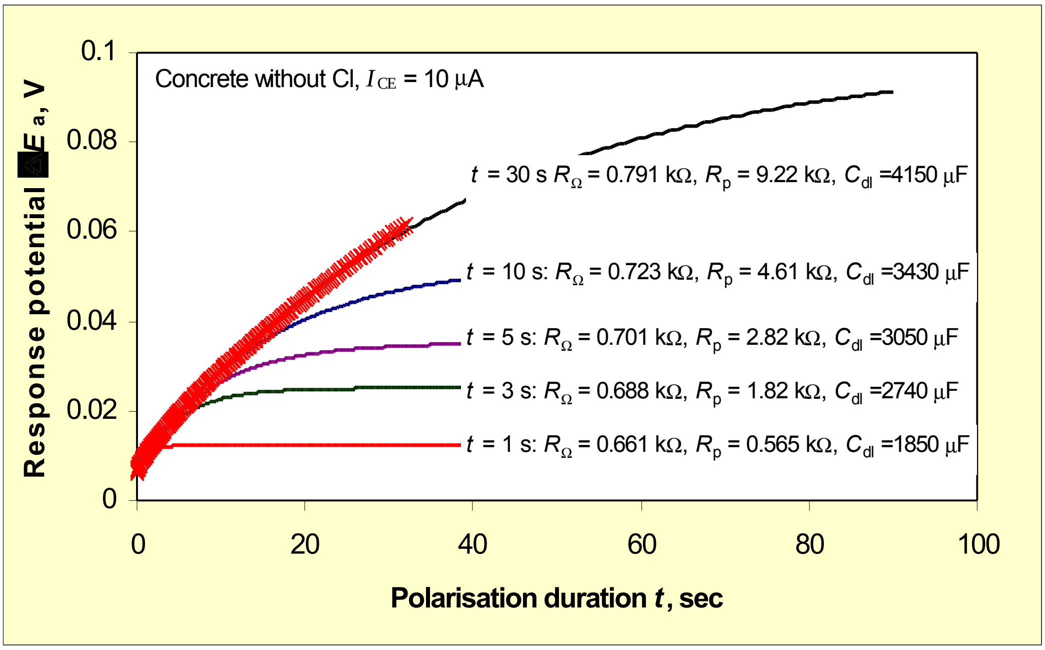



**Figure 5. An example of the curve-fitted results.**

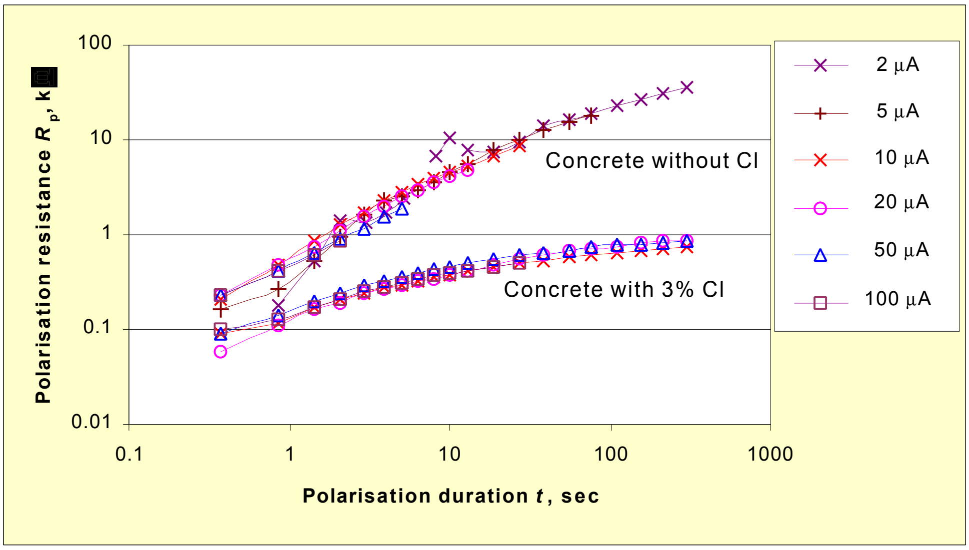



**Figure 6. Polarisation resistances calculated by curve-fitting equation (6) to different durations of each D$E_a$-$t$ curve.**

## 6 DISCUSSIONS

It can be seen from Fig. 6 that the effect of polarisation current appears not significant if the responded potential shift is small enough to assure a linear polarisation. While polarisation duration has tremendous effect on polarisation resistance, no matter if the steel is corroded or not. This phenomenon has also been observed by other researchers (e.g. González et al 1985; Gowers et al 1994; Videm & Myrdal 1997). The effect of sweep rate on polarisation resistance in the dynamic polarisation measurement is in fact the similar phenomenon. A high sweep rate for a certain level of response implies the short polarisation duration, often resulting in a low polarisation resistance (see e.g. González et al 1985; Gowers et al 1994). Figure 6 also shows that the relationships between $R_p$ and $t$ are logarithmically linear, especially when the polarisation duration is in the first few seconds. Therefore, these relationships could be empirically expressed as

$$R_p = at^b \tag{9}$$

where $a$ and $b$ are constants.

At the present the constants *a* and *b* are, however, purely empiric without any physical meanings. It should be born in mind that the above relationship is derived based on the simple Randles model and it may not necessarily mean a zero resistance at $t = 0$ and an infinite resistance at $t \rightarrow \infty$. Gowers et al (1994) also reported the similar relationships between total resistance ($R_p + R_\Omega$) and sweep rate. For passive steel, when sweep rate changed from 0.1 to 1000 mV/min, the total resistance changed from some 10000 to 10 k$\Omega \cdot$cm$^2$! Tang et al (2001) carefully checked the behaviour of ohmic resistance by using the interrupt ("turn-off") technique after a long duration (300 seconds) of polarisation and not observed significant changes in ohmic resistance. Therefore, the change in total resistance should be mainly attributed to the change in polarisation resistance. At the present, the exact mechanisms behind this phenomenon are, however, not clear. Further modelling work is needed to find the convincing explanations to this phenomenon.

The curve-fitted $C_{dl}$ also reveals a similar behaviour, that is, the capacitance increases with polarisation duration, as shown in Fig. 7. The concrete with 3% chlorides gives a higher capacitance than that without chlorides, probably due to its relatively high ionic concentration in the pore solution.

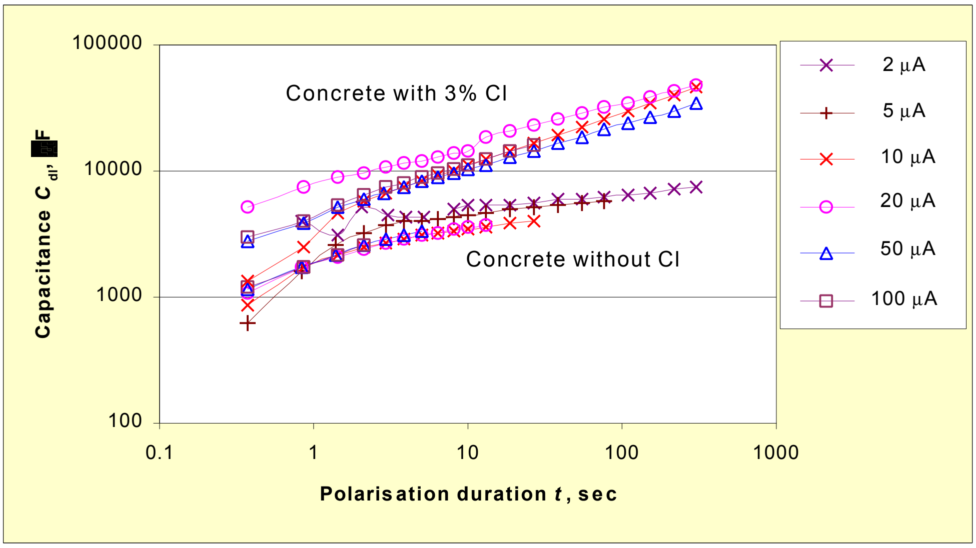



**Figure 7. Double layer capacitances calculated by curve-fitting equation (6) to different durations of each D$E_a$-*t* curve.**

Nevertheless, from the curve-fitted data in Fig. 6 (except for those with fluctuation due to low potential responses), we can calculate the corrosion current density using Equation (1). The calculated results are shown in Fig. 8, where the results measured by two commercial instruments on the same specimens at an early concrete age (4 weeks) are also given. The measurement conditions of different instruments are summarised in Table 1. It can be concluded from Fig. 8 that the differences in corrosion rate measured by different instruments are mainly attributed to different polarisation durations. The challenging question is at which polarisation duration the measured corrosion rate represents the true status of steel in concrete. Since the polarisation technique is an instantaneous measurement of corrosion rate, there is no direct method for immediate calibration of the measured results. The only calibration method is the time-consuming one – a gravimetric method for weight loss of steel, e.g. ASTM G-1. In the literature, the calibration data are very limited (Andrade & Gonzáles 1978; Liu 1996). There is a great need of calibration work before a true value of corrosion rate could be evaluated from the polarisation techniques. A Nordic project is in progress for such a calibration work (Tang et al 2001).

**Table 1. Measurement conditions of different instruments**

| | GECOR | GalvaPulse* | Developed in this study |
|---|---|---|---|
| Polarisation current | Cathodic<br>5~30 µA automatically | Anodic<br>10/100 µA for Cl free<br>100 µA for 3% Cl | Cathodic<br>See Fig. 8 |
| Polarisation duration | 100 sec. | 10/5 sec. for Cl free<br>5 sec. for 3% Cl | See Fig. 8 |
| Guard electrode | Sensor controlled, $\gamma$ = ? | $\gamma$ = 1.1~1.2 | $\gamma$ = 1 |
| Polarised area | 33 cm$^2$ ($L_p$ = 10.5 cm) | 22 cm$^2$ ($L_p$ = 7 cm) | 33 cm$^2$ ($L_p$ = 10.5 cm) |
| *B* constant | 26 mV | 26 mV | 26 mV |
| Reference electrode | $Cu/CuSO_4$ | Ag/AgCl | Ag/AgCl |

measured by T. Frølund, FORCE institute, Denmark.

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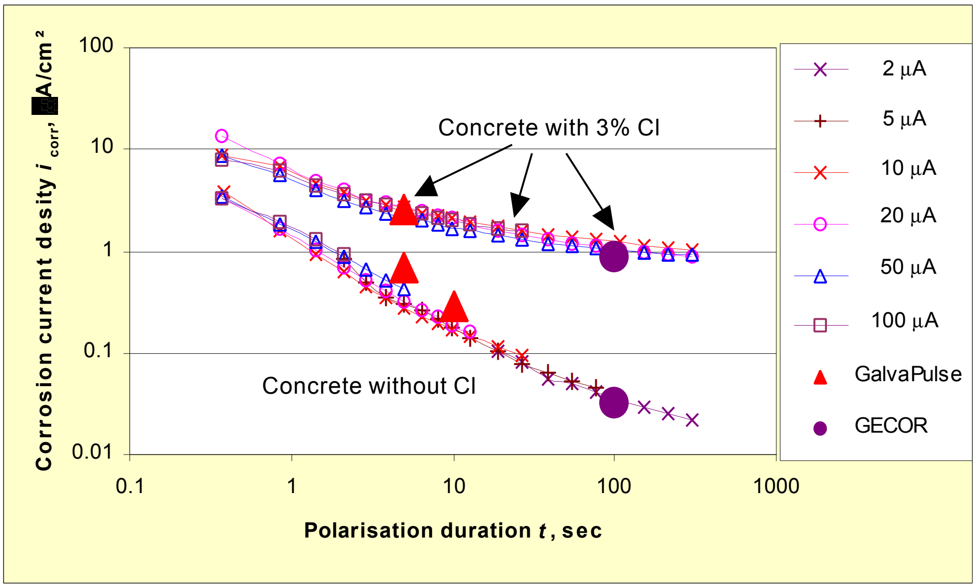


**Figure 8. Corrosion rate measured by different instruments.**

From Fig. 8 it can also be seen that, when the polarisation duration is in a range of 10 to 30 seconds as Polder et al (1994) used in their comparison tests, the difference in corrosion rates is only a factor of 1.5 to 3, which is what Polder et al (1994) found from their results.

## 7 CONCLUDING REMARKS

The following concluding remarks could be made from the results of this study:

- Polarisation duration has tremendous effect on the polarisation resistance calculated based on the simple Randles circuit, no matter if the steel is corroded or not.
- The relationships between polarisation resistance and polarisation duration are logarithmically linear and can be expressed as $R_p = at^b$. At the present the constants *a* and *b* are, however, purely empiric without any physical meanings.
- The effect of polarisation current appears not significant if the responded potential shift is small enough to assure a linear polarisation.
- The double layer capacitance calculated based on the simple Randles circuit reveals a similar behaviour as the polarisation resistance, that is, capacitance increases with polarisation duration.
- The differences in corrosion rate measured by two commercial instruments, GECOR and GalvaPulse, are mainly attributed to different polarisation durations. Calibration work is needed to find proper polarisation duration.

Finally, it should be pointed out that the Randles circuit for the steel-concrete system is only a simplification. The measured $R_p$ with too short polarisation duration may be affected by non-ideal interfacial capacitance, while with long polarisation duration by possible diffusion effect. Possibly the appropriate duration of polarisation for correct determination of $R_p$ should be longer in passive state of steel (larger time constants) than in active state of steel. Further work is needed to find the convincing explanations to the polarisation behaviours of steel-concrete system.

## 8 ACKNOWLEDGEMENTS

The financial supports from the NORDTEST, the SNRA (Swedish National Road Administration) and the FORMAS (Swedish Research Council for Environment, Agricultural Sciences and Spatial Planning) are greatly appreciated. The author thanks Mr T. Frølund at FORCE Institute, Denmark, for supplying the experimental data of GalvaPulse. The author also thanks Dr. P. Fedjstød at ELKEM Materials A/S, Norway, for supplying the instrument GECOR 06 to the Nordtest project.

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# A New Ceramic Tile-Finishing For Renewed External Walls Using Net Overlaying And Anchoring

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**Summary: Mortar plastering and ceramic tiling are the most common building exterior finishings in Japan. In these buildings, when a long time has elapsed from the building completion, the finishings deteriorate according to circumstance.**
**A new renewal method for deteriorated external walls of existing buildings had been developed. The method is applied polyvinylalcohol fiber net and polymer modified cement mortar overlaying stainless steel anchoring. An outline and evaluations of the method have been reported in 7DBMC. At the time of this report, this method has been limited to use in surfaces with a coated finish.**
**Therefore, a new ceramic tile-finishing system for the renewed external walls by fiber net and polymer modified cement mortar overlaying stainless steel anchoring has been developed. In this paper, an outline of subjects, characteristics, materials and application method of the new finishing system are mentioned. And experimental results of quality evaluation in this renewal finishing system are discussed.**

**Keywords: Ceramic tile-finishing, External walls, Net overlaying and anchoring, Renewal method**

## 1 INTRODUCTION

Mortar plastering and ceramic tiling are common building exterior finishings in Japan. In these buildings, when a long time has elapsed from the building completion, the finishings deteriorate according to circumstance.

A new renewal method for treating deteriorated external walls of existing buildings has been developed. The method involves overlaying polyvinylalcohol fiber nets and polymer modified cement mortar with stainless steel pin anchoring. The outline, application method, characteristics of the method and applied materials are mentioned in 7$^{th}$ International Conference on Durability of Building Materials and Components (Kondo *et al.*1996). In addition, evaluations of the quality of the system as a building element for renewal are discussed. The method has been limited to use in surfaces with a coated finish. However, ceramic tile-finishing for external walls is very popular in Japan.

In this background, a new ceramic tile-finishing system for the renewed external has been developed. In this paper, an outline of subjects, characteristics, materials and application methods of the new finishing system are mentioned. Experimental results of a quality evaluation of this renewal finishing system are also discussed.

## 2 OUTLINE OF THE METHOD

### 2.1 Application procedures

Application procedure of the newly developed renewal method for deteriorated existing external walls is shown in Fig.1.

START

Investigation of existing finishing conditions

Repair of surface damage

Surface cleaning by compressed (15-30MPa) water

Surface treatment

Plastering of polymer modified cement mortar (Thickness:2mm)

Overlaying of polyvinylalcohol fiber net

Plastering of polymer modified cement mortar (Thickness:1mm)

Drilling & cleaning holes

Anchoring of specific stainless steel pins

Plastering of polymer modified mortar onto top of the pin

Ceramic tile finishing onto the renewed wall

END

**Figure 1. Standard application procedure of newly developed ceramic-tiling for renewed walls**

### 2.2 Characteristics of the method

Characteristics of the newly developed ceramic tiling method for renewed walls are as follows.

1. The method can make new substrate on which new ceramic tile finishing has kept adhesion strength over $0.4 N/mm^2$.
2. The anchoring pins applied in this method are made from stainless steel SUS 304 with 1400N as the allowable tensile strength per pin. The use of pins for anchoring gives the system outstanding wind load resistance in buildings under 45m.
3. Application of the method has resistance for thermal movement by sunshine and cooling and improves the durability of the external walls.
4. The method can reduce the waste volume which occurs during application, and it shortens the construction term.

### 2.3 Materials for the method

Some materials used in this method are shown below.

*Polymer modified cement mortar*

In this method, the polymer modified cement mortar containing cationic styrene-butadiene latex type polymer is applied.

The modified cement mortar was evaluated based on JIS A 6916$^{-1995}$ (Surface preparation materials for coatings). Table 1 shows test results by specimen, which were prepared in proportion to the main material : mixture liquid = 25:6 in weight according to its manufacturer's specification.

**Table 1. Properties of polymer modified cement mortar**

| Items | Characteristics | Criteria in JIS A 6916 |
|---|---|---|
| Fluidity change (%) | 1.1 | +20 to -20 |
| Adhesion strength (N/mm2) | | |
| Curing at 20 degrees C. | 1.5 | Over 1.0 |
| Curing at 3 degrees C. | 1.2 | Over 0.7 |
| Cracking | None | None |
| Impact resistance | Non cracking & delamination | Non cracking & delamination |
| Water absorption (g) | 0.7 | Under 1.0 |
| Durability *; Appearance | Non cracking, cracking & delamination | Non cracking, cracking & delamination |
| Adhesion strength (N/mm2) | 1.3 | Over 1.0 |

* Notes Durability test method: 10 times of 18 hour-dipping at room temperature, 3 hour-cooling at -20 degrees C. and 3 hour-heating at 50 degrees C.

*Fiber net*

Tri scrim twisted polyvinylalcohol fiber net is used for facial continuation and reinforcement. Properties of the material have already been shown in 7DBMC (Kondo *et al.*1996).

*Anchoring pins*

Properties of the anchoring pins that were applied in this method have already been shown in 7DBMC (Kondo *et al.*1996).

Based on pull-off test results of the anchoring pins, 1400N average maximum load minus 3. (standard deviation) is adopted as the allowable tensile strength per pin.

### 2.4 Evaluation of the method

**Adhesion of the existing finishing**

*Polymer modified mortar onto ceramic tile surface*

<u>Experiments.</u> Substrates: Ceramic tiling with a gloss or non-gloss surface onto concrete plate

Preparation: Polishing with sandpaper for tile surface or non-polishing

Application: Priming of ethylene vinyl acetate emulsion or non-priming and plastering of polymer modified mortar (thickness:3mm)

Adhesion test:The test was conducted 7, 14 or 28 days after the application of specimens. An oil-pressure type apparatus was used for the pull-off test.

Results. Results of the adhesion test are shown in Table 2.

**Table 2. Adhesion of the method to the existing ceramic tile**

| Tile surface | Surface preparation | Priming | 7 day-age (N/mm2) | 14 day-age (N/mm2) | 28 day-age (N/mm2) |
|---|---|---|---|---|---|
| Gloss | P | P | 2.3 | 2.6 | 2.7 |
| Gloss | P | N | 2.5 | 2.7 | 3.2 |
| Gloss | N | P | 2.8 | 3 | 3.2 |
| Gloss | N | N | 2.9 | 3.1 | 3.2 |
| Non-gloss | P | P | 2.3 | 2.8 | 3.3 |
| Non-gloss | P | N | 2.5 | 3 | 3.1 |
| Non-gloss | N | P | 2.5 | 3 | 3.1 |
| Non-gloss | N | N | 2.8 | 2.9 | 3.4 |

Notes

Tile surface:Gloss means glossy surface tile

Non-gloss means dim surface tile

Surface preparation:N means non-polishing

P means polishing

Priming:N means no application

P means primer application

Failure in adhesion test:Occured a mostly inside the polymer modified cement mortar

Adhesion strength:Average of the test of 3 specimens is shown

*Polymer modified mortar onto various kinds of coating material surfaces*

Experiments. Substrates:3 kinds of coating finishes

E:Synthetic resin emulsion type wall coating for waterproofing

R:Reactive synthetic resin emulsion type wall coating

W: Reactive synthetic resin emulsion type wall coating for waterproofing

Application:The substrate was left for 4 weeks after the application of the above coating finishes inside a room at a temperature of 20 degrees C and 60% RH.

Adhesion test:The test was conducted 2 weeks after the application to above specimens. An oil-pressure type apparatus was used for the pull-off test.

Results. Results of the adhesion test are shown in Table 3.

Failure in substrate E test occurred inside of coating material. And in case of R test, the failure was in the polymer modified mortar. In case of substrate W test the failure occurred inside of coating material and interface between coating and polymer modified mortar.

**Table 3. The adhesion of the method onto the existing coating finishes**

| Coating | Priming | Adhesion strength |
|---|---|---|
| E | P | 0.8 |
| | N | 0.78 |
| R | P | 2.6 |
| | N | 2.2 |
| W | P | 0.95 |
| | N | 0.96 |

($N/mm^2$)

Notes

Priming:N means no application

P means primer application

Adhesion strength:Average of the test of 5 specimens is shown in $N/mm^2$.

### 2.5 Evaluation as a building element

Renewal systems for external walls have to possess elasticity because external walls treated with this renewal method undergo some movement with heat change and deformtion by earthquakes. Therefore, an evaluation of this method as a building element is demanded.

*Delamination by repeated heating and cooling*

Experiment. Specimens: Concrete $Fc=24N/mm^2$ Size 900* 1800* 150mm

Curing 1 month at room temperature

Application: Mortar plastering and tile finishing or coating finishing

This renewal method and new tile finishing was applied onto the substrate 2 weeks after the existing finishing.

Heat cycle: The heating and cooling test was repeated 2 weeks after the specimen's production. The heating was stopped when the specimen's surface had reached 70 degrees C, then air-cooling continued for 8 to 12 hours until the specimen's surface had reached 15 degrees C. This heating and cooling was repeated 10 times.

Results. The main results of this experiment are as follows.

Adhesion strength following this experiment was over $0.6N/mm^2$, which is generally considered sufficient strength. Furthermore, no deterioration by movement with heat had been observed.

*Deformation by substrate deformation*

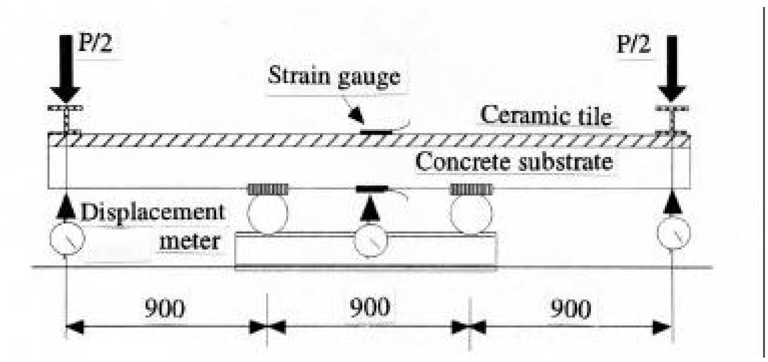
Experiments. Specimens: Concrete $Fc=24N/mm^2$ Size 1000x2800x150mm

Curing 1 month at room temperature

Application: Mortar plastering and tile finishing or coating finishing

This renewal method and new tile finishing was applied onto the substrate 2 weeks after the existing finishing.

Bending test: Specimens for the bending test was conducted 2 weeks after the specimen's production. The test method is shown in Figure 2.




(mm)

**Figure 2. Bending test method**

Results. The main results of this experiment are as follows.

The specimens are monolithic for transverse deformation of substrate. It is judged that adhesion strength between the existing finishing layer and the newly renewed substrate layer is enough for practical purposes as they are anchored together.

### 2.6 Wind load resistance

Theoretical consideration of wind load resistance was based on 'Recommendations for Loads on Buildings' by the Architectural Institute of Japan. For the design, the wind velocity of 38m/s was adopted which is the typical value in Japanese major cities. Building height was estimated to be under 45m. According to these recommendations, the calculation results are as follows. The wind load on general areas of the building was $993N/m^2$ and on corners of the building the wind load increased to $1986N/m^2$. The allowable tensile strength per pin is 1400N and the safety factor for temporary loads is 1.67. The temporary wind load was 838N which is 1400 divided 1.67.

Go Back

Therefore, the required number of anchoring pins in the system is as follows. The number of pins necessary for general areas of a building is 1.19 and in building corners it is 2.37. Actually, the system uses 2 pins in general building areas and 3 pins in corners. The system has enough safety wind load resistance by application of these pins.

### 2.7 Ceramic tile application method

An organic elastic adhesive is adopted as the ceramic tile application method in this renewal system. Evaluation of the tiling with organic elastic adhesive is described in 8$^{th}$ International Conference on Durability of Building Materials and Components (Kondo *et al.*1999).

## 3 CONCLUSION

A new renewal method for treating deteriorated external ceramic tiling and mortar plastering walls of existing buildings has been developed. This renewal method involves overlaying polyvinylalcohol fiber nets and polymer modified cement mortar with stainless steel anchoring. Also, new ceramic tile finishing onto the renewed walls is included.

In this paper, the performance of the materials which are applied in this renewal method are introduced and it is confirmed that they are suitable. In addition, this system is evaluated as a building element and it is judged that this method is superior to renovated external walls.


## 4 ACKNOWLEDGMENTS

This study was carried out by a research committee of the Association of Building Research Promotion. The authors would like to express their heartfelt thanks to other research members.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# Correlation Of Adhesive Strength With Service Life Of Paint Applied To Weathered Wood

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**Summary: Smooth-planed western redcedar bevel siding was exposed outdoors (preweathered) for 1, 2, 4, 8, and 16 weeks. The weathered boards were separated into two end-matched groups. One group was painted with only primer paint and tested for paint adhesion following preweathering. The second group was painted with a primer and topcoat, exposed outdoors for an additional 17 years, and evaluated during this period for paint cracking and flaking. There was a direct correlation between the amount of time the siding was preweathered and the long-term paint performance. Those boards preweathered for 16 weeks began to show cracking after about 3 years, whereas those boards that were not preweathered were in almost perfect condition after 17 years. Paint adhesion tests done on end-matched boards that were preweathered at the same time and then painted showed good correlation between the paint bond strength and outdoor paint performance for the preweathered boards.**

**Keywords: Alkyd-based paint, latex-based paint, weathering, western redcedar, paint adhesion.**

## 1 INTRODUCTION

In the absence of adhesion failure, paint on wood exposed outdoors gradually erodes. Degradation of paint by erosion may take a decade or more, depending on the degree of exposure to sunlight and moisture and the thickness and type of paint. While a paint system is eroding, it still protects the wood surface from degradation. Until this erosion process proceeds to the point where the primer begins to show, the paint surface can be renewed readily with an additional topcoat. With timely refinishing, painted wood can last for centuries (Feist & Hon 1984).

If, however, the paint–wood interphase fails, the paint film will debond within a short time and the paint will blister, crack, and peel. This failure can result in damage to the wood surface and more difficult and costly refinishing. One cause of interphase failure is a degraded wood surface caused by weathering prior to initial priming with paint (Arnold *et al.* 1992, Boxall 1977, Bravery & Miller 1980, Desai 1967, Evans *et al.* 1996, Kleive 1986, Miller 1981, Shurr 1969, Thay & Evans 1998, Underhaug *et al.* 1983, Williams & Feist 1994, Williams *et al.* 1990). These previous studies have also shown that weathering of wood prior to painting (preweathering) decreases subsequent paint performance. However, the amount of preweathering has not been quantitatively linked to long-term paint performance. For example, no study has looked at how short periods of preweathering (one week, several weeks, a few months, several months) affect long-term paint performance (more than 10 years).

This study reports the results of paint adhesion tests on newly painted preweathered boards and paint performance after 17 years of outdoor exposure on boards that were similar to those used for the paint adhesion tests and that were also preweathered for the same amounts of time. The results clearly show the effect of short periods of preweathering (1 to 16 weeks) on the performance of three different paint systems (two different primers) exposed outdoors for 17 years. Paint performance is then compared with the adhesive tests previously performed on end-matched boards that were preweathered the same amount, then painted using the same two primers, but were not exposed outdoors after painting. The paint adhesion test results and paint performance after 14 years were reported in more detail earlier (Williams *et al.* 1987, Williams & Feist 2001).

## 2 EXPERIMENTAL

### 2.1 Materials

The finishes were applied to smooth-planed western redcedar (WRC) (*Thuja plicata* Donn) vertical-grained heartwood. The boards for the paint adhesion tests were finished with either two coats of alkyd-oil primer or two coats of latex primer, and the boards for the outdoor exposure were finished with either (1) solventborne water-repellent preservative (WRP), one coat of alkyd-oil primer, and one coat of acrylic latex topcoat (WRP/alkyd/latex); (2) one coat of alkyd-oil primer and one coat of acrylic latex topcoat (alkyd/latex); or (3) one coat of latex primer and one coat of acrylic latex topcoat (latex/latex). All finishes were commercial formulations. For each of the preweathering periods (0, 1, 2, 4, 8, and 16 weeks), 12 boards were exposed outdoors for 17 years.

### 2.2 Methods

Freshly planed vertical-grained WRC boards 410 by 100 by 10 mm (16 by 4 by 3/8 in.) (longitudinal by radial by tangential) were exposed outdoors, oriented vertically facing south 15 km west of Madison, Wisconsin, in the summer of 1984 for 1, 2, 4, 8, or 16 weeks. At the same time, controls (0-week specimens) were kept from exposure to sunlight in a darkened room at 27? C and 65% relative humidity for 16 weeks. Following weathering, the WRC boards were lightly washed with distilled water, air-dried, and painted. The boards were randomly divided into two groups. One group was finished with two coats of primer and was used to conduct paint adhesion studies. One half of each board used for paint adhesion tests was painted with alkyd-oil primer and the other half with acrylic latex primer. The second group was finished with the paint systems described in the previous section and placed back on the test fence in September 1984. Boards from all preweathering periods (1, 2, 4, 8, and 16 weeks) were used for the WRP/alkyd/latex paint system. Only boards preweathered for 0-, 1-, and 16-weeks were finished with the other two paint systems (alkyd/latex and latex/latex).

Boards for the adhesion tests were cured for 3 months, then freshly planed hard maple (*Acer saccharum*) boards were glued to the painted surfaces using an emulsion polymer/isocyanate (EPI) adhesive. The resulting panels were cured in a press at 520 kPa (75 lb/in$^2$) at room temperature for 36 hours. Tensile specimens and block shear specimens were cut from each assembled WRC/maple panel after the adhesive cured. Both had 25- by 25-mm (1- by 1-in.) bond areas. The tensile specimens were then glued to aluminum blocks **(**Fig. 1) using an epoxy/polyamide adhesive and were cured for 48 hours at room temperature.

Expanded cross sections of both the tensile and the shear specimens (Fig. 1) show several interphases: wood/paint, paint/EPI, and EPI/maple. In addition, the final tensile specimens had wood/epoxy and epoxy/aluminum interphases. Hard maple, being a stronger wood, shifted the failure toward the weaker WRC/paint interphase or to the WRC. The shear specimen was a further-modified version of the specimens as described in American Society for Testing and Materials (ASTM) D905 (ASTM 1981) and modified by Strickler (1968).

Tensile and shear specimens were subsequently equilibrated to 12% equilibrium moisture content (EMC) and tested at a constant-displacement load rate of 1 mm/min and 0.38 mm/min, respectively. Load and deflection readings were acquired during each tensile or shear test. Ultimate stress and the elastic stress-strain modulus were calculated from these values. Failure of the paint/EPI, EPI/maple, wood/epoxy, or epoxy/aluminum interphases was deemed unacceptable because only failures of the weathered wood substrate or of the WRC/paint interphase were considered pertinent. Accordingly, all specimens were visually examined for failure following testing, and only those exhibiting the specified failure type were used to compare adhesion.

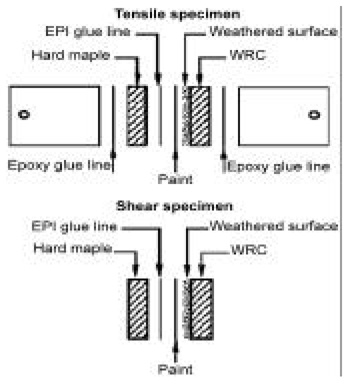


**Figure 1. Expanded view of tensile and shear specimens showing interphases (ML86-5258)**

For outdoor exposure, three boards were mounted together to form a panel configured as lap siding. Four panels were tested for each preweathering time. The boards were evaluated annually according to ASTM standards for erosion (ASTM 1991a), cracking (ASTM 1991b), and flaking (ASTM 1991c). Each board in the panel was rated individually, resulting in 12 observations for each category (flaking and cracking), annually or biannually for 17 years. A rating of 10 indicates no observable degradation, and 1 indicates complete failure of the specimen. A rating of 5 indicates sufficient degradation to warrant normal refinishing if the finish was in use on a structure.

## 3 RESULTS AND DISCUSSION

### 3.1 Adhesion tests: latex primer

#### 3.1.1 Tensile tests

Many specimens weathered less than 4 weeks before painting failed within the WRC substrate, and this was attributed to cohesive failure in the wood and not to weathering. This wood failure occurred away from the interphase at a depth of 2 to 3 mm and therefore was not caused by weathering because sunlight penetrates the wood surface only about 75 µm (Hon & Ifju 1978). Specimens weathered for 8 or 16 weeks failed almost exclusively at the paint/wood interphase.

A plot of all failures in the tensile tests of latex primer is shown in Fig. 2. A Duncan multiple range test of means (Duncan 1955) showed no difference between controls and specimens exposed for 1, 2, and 4 weeks. The distribution in tensile strengths from 0 to 4 weeks (Fig. 2) is probably attributable to wood variation not paint adhesion. Mean tensile strength remains constant for up to 4 weeks, then as interphase failure becomes the dominant failure mode, it begins to decline. This trend can be more easily seen when specimens that failed totally in the wood are deleted (Fig. 3). The mean tensile strength of the wood/latex primer bond decreased from 2.1 MPa (310 lb/in$^2$) after weathering for 4 weeks to 1.0 MPa (150 lb/in$^2$) after weathering for 16 weeks (Table 1).

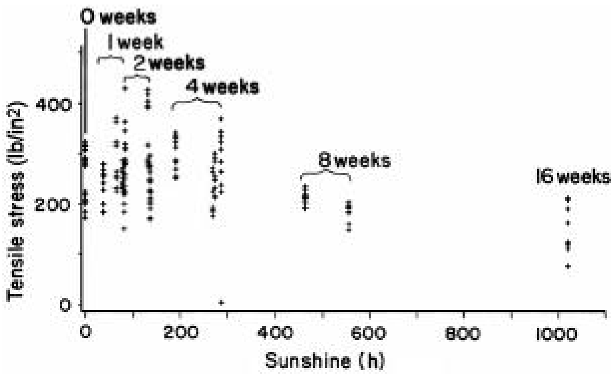


**Figure 2. Ultimate tensile stress compared with sunlight exposure time of acrylic latex primer on WRC. All adhersive failures and cohesive wood failures are shown (1 lb/in$^2$ = 6.9 kPa) (ML86 5259).**

The mean tensile and shear strength at failure for both paints are listed in Table 1. Using a linear model, a Duncan's multiple range test of means shows significant (alpha = 0.05) loss of adhesion for all groups after 4 weeks of weathering. A value of 0.05 indicates 95% confidence that there is a significant difference between two means. This is shown by the breaks in the underlines in Table 1. Load-deflection curves were plotted for all tests. Latex primer exhibited a greater overall deflection prior to failure, lower modulus of elasticity, and higher adhesive strength than did the oil primer. This probably relates more to physical differences between the two paints than to weathering effects.

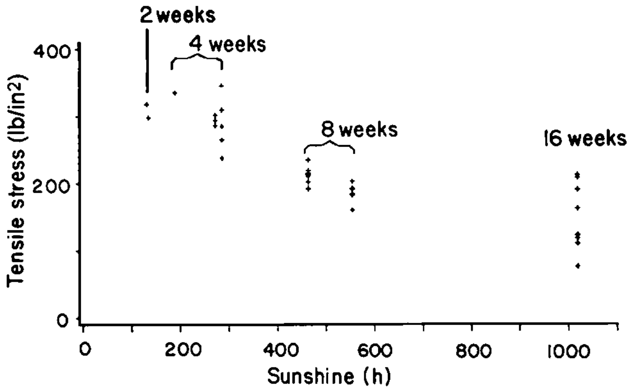


**Figure 3. Ultimate tensile stress compared with sunlight exposure time of acrylic latex primer on WRC. Only primer/wood interphase failures are shown (1 lb/in$^2$ = 6.9 kPa) (ML86 5260).**

**Table 1—Results of a Duncan multiple range test on mean adhesive strength of wood/primer at Alpha = 0.05[a]**

| | Tensile test | | | | | | Shear test | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Amount of preweathering | | | | | | Amount of preweathering | | | | | |
| | 0 | 1 week | 2 weeks | 4 weeks | 8 weeks | 16 weeks | 0 | 1 week | 2 weeks | 4 weeks | 8 weeks | 16 weeks |
| Latex primer[b] | | | | | | | | | | | | |
| Number flaking[c] | 0 | 0 | 2 | 10 | 15 | 10 | 24 | 11 | 24 | 24 | 12 | 6 |
| Strength (lb/in$^2$) | — | — | 310 | 305 | 200 | 150 | 800 | 765 | 750 | 710 | 560 | 450 |
| Strength (MPa) | — | — | 2.1 | 2.1 | 1.4 | 1.0 | 5.5 | 5.3 | 5.2 | 4.9 | 3.9 | 3.1 |
| Alkyd-oil primer[d] | | | | | | | | | | | | |
| Number flaking[c] | 0 | 0 | 19 | 14 | 18 | 6 | 7 | 0 | 15 | 22 | 12 | 6 |
| Strength (lb/in$^2$) | — | — | 190 | 255 | 155 | 125 | 690 | — | 700 | 675 | 530 | 490 |
| Strength (MPa) | — | — | 1.3 | 1.8 | 1.1 | 0.87 | 4.8 | — | 4.8 | 4.7 | 3.7 | 3.4 |

[a]Strength values underlined by the same line are not significantly different, with 95% confidence.
[b]For latex primer, $R^2$ = 0.782 in tensile test and $R^2$ = 0.591 in shear test.
[c]Number of specimens flaking at the paint/wood interphase.
[d]For alkyd-oil primer, $R^2$ = 0.579 in tensile test and $R^2$ = 0.455 in shear test.

#### 3.1.2 Shear tests

In the shear test, there was little wood substrate failure because failures occurred primarily at the latex primer/wood interphase with essentially 100% primer adhesion failure on the 8- and 16-week specimens (Fig. 4). The shear results were similar to the tensile results and showed no significant differences in mean shear strengths of 5.5, 5.3, and 5.2 MPa (800, 765, and 750 lb/in$^2$), respectively, for specimens exposed for 0, 1, or 2 weeks. The 4-week specimens were statistically different than the controls but not different than the 1- and 2-week specimens (Table 1). The decrease in adhesion after 4 weeks of exposure is evident in Fig. 5. The decrease in strength for the controls from 5.5 MPa (800 lb/in$^2$) to 3.1 MPa (450 lb/in$^2$) after 16 weeks was not as great as with the tensile values (Table l). However, the trend was the same. As with the tensile tests, failure at the primer/EPI and the EPI/maple interphases was ignored and only the results from specimens that failed at the wood/primer interphase were plotted.

### 3.2 Adhesion tests: alkyd-oil primer

#### 3.2.1 Tensile tests

The mean tensile strength of the oil primer on wood weathered 4 weeks before painting was 1.8 MPa (255 lb/in$^2$) compared with 870 kPa (125 lb/in$^2$) after 16 weeks of weathering (Table 1). As with the latex primer, ultimate strength for many specimens weathered 2 weeks or less reflected only wood failure and were deleted. The failure mechanism for the oil primer is more complicated than for the latex primer because adhesion of paint to latewood (summerwood) is better than to earlywood (springwood) (Fig. 4). This failure of the earlywood/paint interphase rather than the latewood/paint interphase is opposite to the expected failure site.

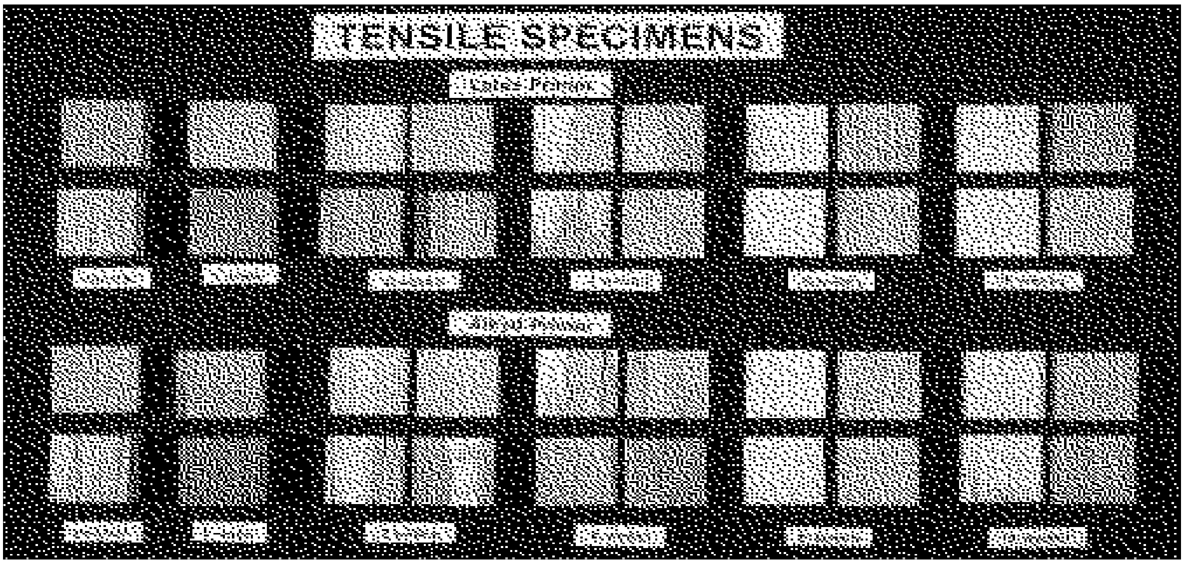


**Figure 4. Failure surfaces of representative tensile specimens (M86 0043).**

With both paints, the differences in the elastic modulus of earlywood and latewood bands in wood may set up stress concentrations at the junction of these bands. The flexible latex primer film may more easily absorb this differential strain energy without failing. The less flexible oil primer cracks at the strain energy levels along the earlywood/latewood boundaries, failing at lower loads than the latex primer. This type of crack formation of oil primer at the earlywood/latewood boundary has previously been reported (Miniutti 1965, 1974). Differences in earlywood and latewood primer adhesion of the oil primer may be related to failure at this boundary. The change in adhesive strength with weathering was less for the alkyd-oil primer than for latex primer. This may be caused by the higher adhesive strength to the latewood. However, as mentioned, the greater adhesion of the oil primer to latewood was unusual because it is fairly well accepted that paint adheres better to earlywood. Apparently, this traditional view of better paint adhesion to earlywood is appropriate only for unweathered wood.

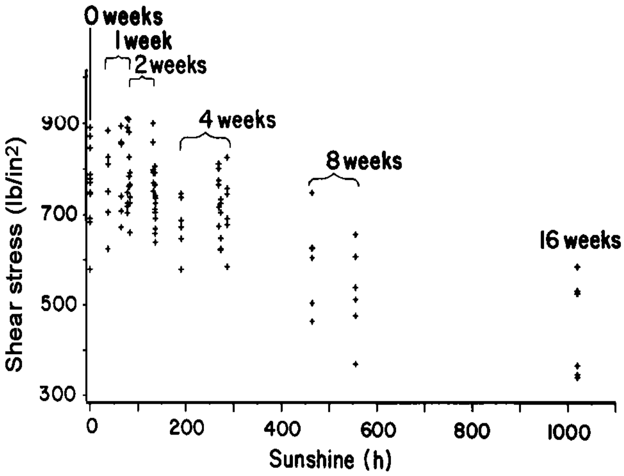


**Figure 5. Ultimate shear stress compared with sunlight exposure time of acrylic latex primer on WRC. Only primer/wood interphase failures are shown (1 lb/in$^2$ = 6.9 kPa) (ML86 5261).**

The results of these experiments showed no difference between earlywood and latewood adhesion for the controls and the 1-week specimens. After a short period of weathering, however, the damage to the earlywood is sufficient to cause paint failure on this part of the substrate. Although only a few of the 2-week-weathered specimens failed at the earlywood/paint interphase, this apparent anomaly became the general failure site in the specimens weathered 4, 8, and 16 weeks before painting (Fig. 4). The differential failure of the oil primer on earlywood/latewood boundary and the uniform failure of the latex paint may be explained by the difference in the interphase formed by these different paints with the weathered wood surface.

#### 3.2.2 Shear tests

The change in shear strength of oil primer with time shows the same trend as the tensile results. The mean adhesion strength dropped from 4.8 to 3.4 MPa (700 to 490 lb/in$^2$) between no preweathering and 16 weeks of preweathering (Table 1). As observed in the tensile tests, the shear specimens showed that the oil primer adhered stronger to weathered latewood than did the latex primer.

### 3.3 Outdoor weathering of paint: cracking and flaking

The most notable differences among the finishes were found for cracking and flaking. The effect of preweathering on these paint degradation mechanisms were evaluated for 17 years for the three different paint

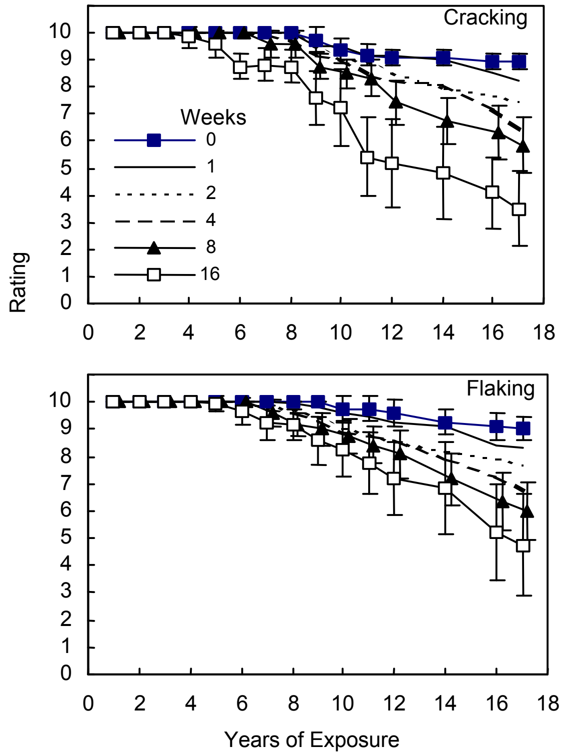


**Figure 6. Paint evaluations for cracking and flaking during 17 years for the paint system of solventborne water-repellent preservative, one coat of alkyd-oil primer, and one coat of acrylic latex topcoat (WRP/alkyd/latex). The data points are the average of 12 observations, and the bars give the standard deviation.**

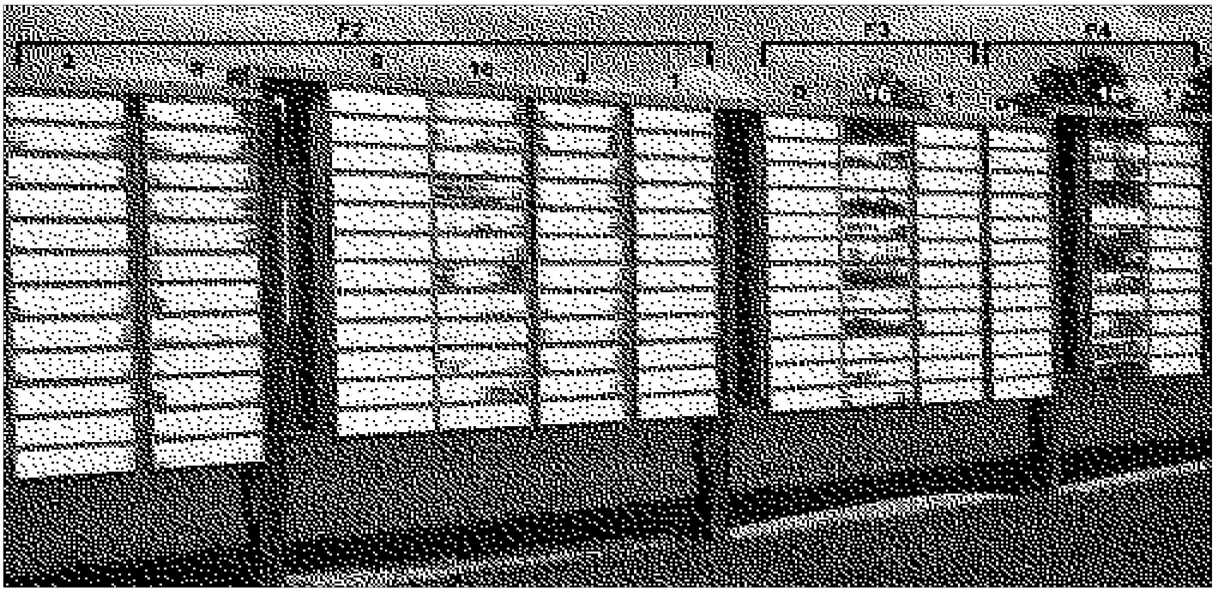

**Figure 7. Exposure fence west of Madison, Wis., showing painted specimens after 17 years of outdoor exposure (F2, WRP/alkyd/latex; F3, alkyd/latex; F4, latex/latex; numbers are weeks of preweathering).**

systems. In addition to the effect of preweathering on paint degradation, the experimental design also included the effects of a WRP pretreatment and an oil-alkyd versus a latex primer. Depending on the amount of preweathering, the boards painted with any of the three paint systems began to show cracking during the exposure period. Flaking generally followed cracking after a year or two (Fig. 6). Differences in paint performance after 17 years of exposure for the different preweathering periods can clearly be seen in Figure 7. Each vertical section contains 12 replicates for the different preweathering periods. For the three paint systems, boards that were not preweathered are in excellent condition, whereas boards that were preweathered for 16 weeks have failed. There is considerable variation among the 12 replicates of the 16-week preweathered boards, but the trends are obvious.

### 3.3.1 WRP/alkyd/latex paint system

The effect of preweathering can clearly be seen in the cracking and flaking evaluations during the 17 years. The boards with 16 weeks of preweathering began to show signs of cracking after only 3 years of exposure, whereas those with 0 and 1 week of preweathering began to crack after 9 years. Clearly, each preweathering period had different performance results. This difference in performance can be seen in the photographs of the boards after 17 years of exposure (Fig. 8a–f). One panel (3 of the 12 boards) for each of the preweathering times is shown. The other three panels showed the same trend. Although it is not apparent in cracking results in Figure 6, there is clearly a slight difference in performance between the 0- and 1-week preweatherings (Fig. 8a and b). The control (0-week preweathering) was in almost perfect condition after 17 years of exposure. That is a service life of more than 17 years for a paint system comprised of a WRP pretreatment, one coat of primer, and one topcoat. The slight discoloration just under the bottom edge of each board is dirt.

### 3.3.2 Alkyd/latex paint system

The cracking and flaking ratings for the 16-week preweathering periods are slightly lower for the alkyd/latex paint system without the WRP (Fig. 9) than for the specimens with the same amount of preweathering finished with the WRP/alkyd/latex paint system (Fig. 8). This can be seen by comparing photographs of panels from the different paint systems preweathered for the same amount (Fig. 10a, b, and c compared with Fig. 8a, b, and f). After 17 years, there was a slight improvement in paint performance in boards pretreated with a WRP compared with those without the pretreatment, particularly for boards that had been preweathered. There was no apparent difference for the control boards. Paint cracking developed more quickly on boards without WRP. Also there appears to be a slight difference between the 0- and 1-week preweathering periods for boards without the WRP pretreatment (Fig. 9). For WRP-treated boards, flaking occurred about 1 to 3 years after cracking. However, for boards without WRP, flaking was immediately evident upon cracking.

### 3.3.3 Latex/latex paint system

In general, ratings for the performance of the alkyd/latex paint system (Fig. 9) were slightly higher than those of the latex/latex paint system (Fig. 11). There was clearly a difference between the two paint systems within the 0-, 1-, and 16-week preweathering periods. Both the alkyd/latex (Fig. 9) and the latex/latex (Figs. 11 and 12) paint systems started cracking and flaking about the same time (after 3 to 4 years of exposure), but the latex/latex system degraded faster in subsequent years. For the latex/latex paint system, the paint on the 0-week preweathered boards cracked and flaked sooner than expected given the inherently greater flexibility of the latex/latex paint system compared with the alkyd/latex system.

## 3.4 Paint adhesive strength compared with cracking and flaking evaluations

Figure 13 shows the average adhesive strength of the alkyd-oil primer compared with the cracking or flaking evaluations for the WRP/alkyd/latex paint system for specimens preweathered for 2 to 16 weeks. There appears to be a good correlation even though there was considerable cohesive wood failure in the specimens preweathered for only 0, 1, and 2 weeks. The tensile strength of WRC perpendicular to grain is about 1.5 MPa and the shear strength parallel to grain is about 6.8 MPa (Forest Products Laboratory 1999, table 4-3a). Therefore, the regression analysis included only data pertaining to paint/wood bond strength. Even those specimens preweathered for 2 weeks had sufficient paint adhesive strength to cause primarily wood failure in the adhesion tests. This was less of a problem with the shear tests because the shear strength of WRC parallel to grain was somewhat higher than the paint/wood shear strength for the 2-week preweathered specimens.

If any part of the paint bond was visible after the adhesion test, the specimen was included in the data set. In Fig. 13c and d (tensile strength versus cracking and flaking), the abrupt drop for the 2-week preweathered specimens is probably caused by a failure in the wood. The $R^2$ values for the comparisons of shear strength with cracking or flaking were 0.81 and 0.98, respectively. The $R^2$ values for the comparisons of tensile strength with cracking or flaking were 0.61 and 0.56, respectively. If the 2-week preweathering data are removed from the data set, the $R^2$ values are 0.73 and 0.97 for cracking and flaking, respectively Thus, it appears that paint adhesion tests give a reasonable indication of long-term performance of paint. Paint performance without cracking and flaking seems to require a shear strength of at least 5.0 MPa and a tensile strength of 1.8 MPa on smooth-planed WRC.

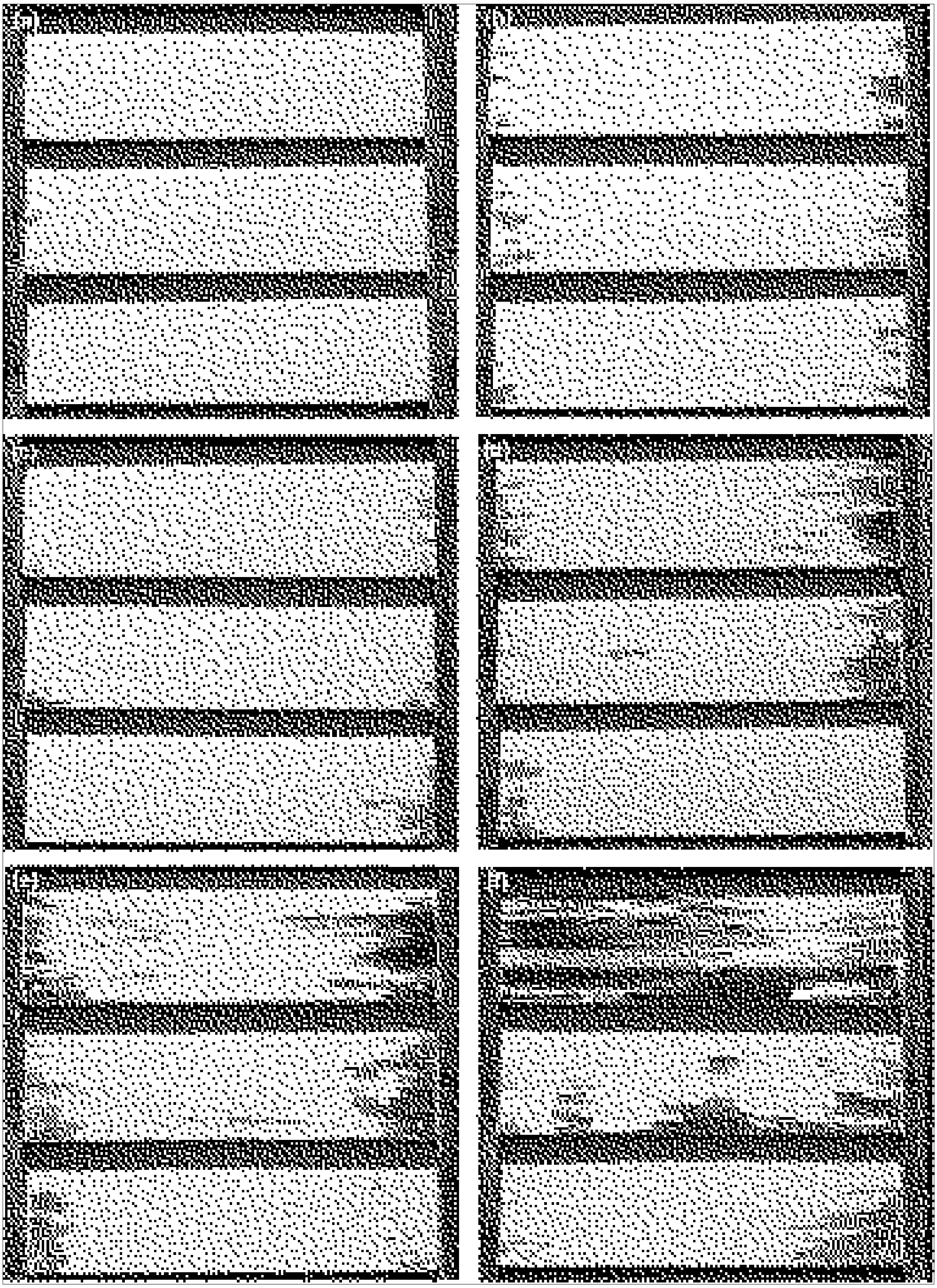

**Figure 8. Examples of panels painted with a solventborne water-repellent preservative, one coat ofalkyd-oil primer, and one coat of acrylic latex topcoat (WRP/alkyd/latex) after 17 years of outdoorexposure. (a) control, no exposure prior to painting, (b) preweathered 1 week, (c) preweathered 2 weeks, (d) preweathered 4 weeks, (e) preweathered 8 weeks, and (f) preweathered 16 weeks.**

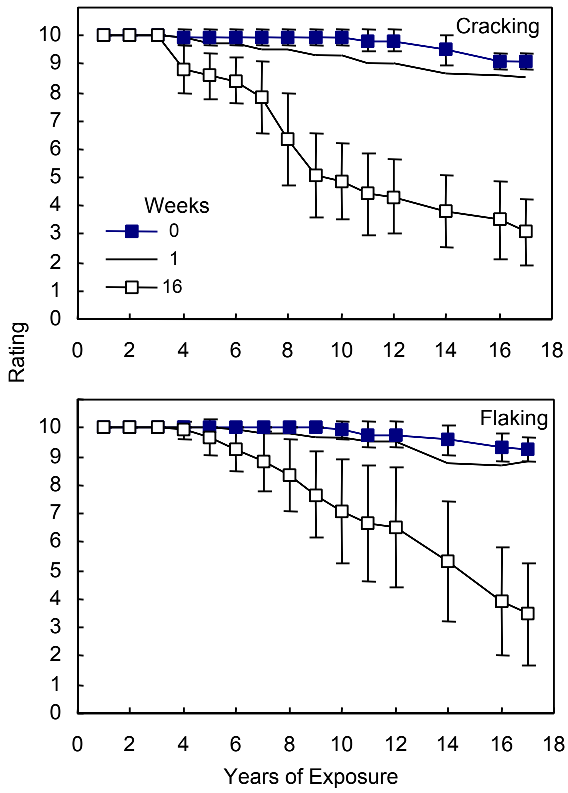


**Figure 9. Paint evaluations for cracking and flaking during 17 years of outdoor exposure for specimens painted with one coat of alkyd-oil primer and one coat of acrylic latex topcoat (alkyd/latex). The data points are the average of 12 observations, and the bars give the standard deviation.**

## 4 CONCLUSIONS

The exposure of unpainted smooth-planed, vertical-grained WRC siding to weather for as little as 1 to 2 weeks can shorten the service life of subsequently applied paints. For wood exposed unfinished for 16 weeks prior to painting, cracking in the paint film was detected after only 3 years of outdoor exposure. In contrast, boards that were not exposed to the weather prior to painting were in almost perfect condition after 17 years of exposure. Paint adhesion tests gave a good indication of service life for those specimens exposed 4 or more weeks prior to painting.

However, the adhesion tests did not indicate potential problems with cracking and flaking for specimens preweathered for short periods because the paint/wood bond strength was about the same as the wood strength. The outdoor performance of painted wood that had been preweathered for short periods showed that there was undoubtedly some surface degradation of these specimens caused by the preweathering. It is imperative that smooth-planed lumber be painted promptly during construction.

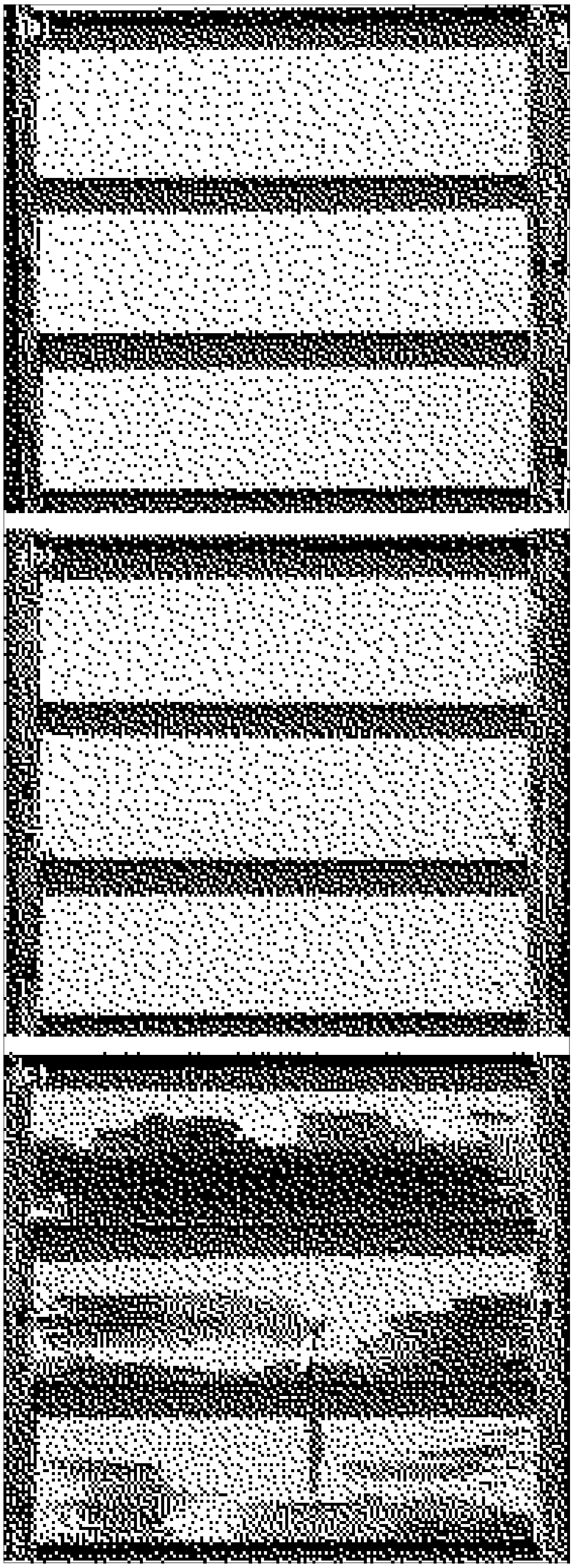

**Figure 10. Examples of panels painted with one coat of alkyd-oil primer and one coat of acrylic latex topcoat (alkyd/latex) after 17 years of outdoor exposure. (a) control, no exposure prior to painting, (b) preweathered 1 week, and (c) preweathered 16 weeks.**

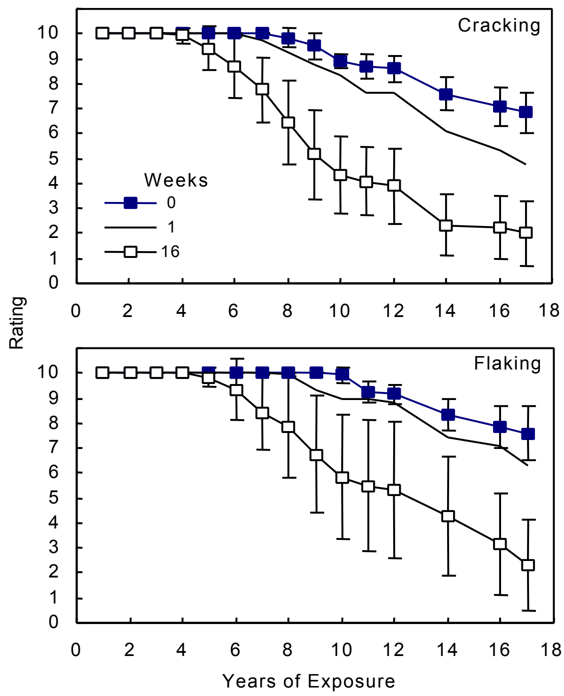


Figure 11. Paint evaluations for cracking and flaking during 17 years of outdoor exposure for specimens finished with one coat of latex primer and one coat of acrylic latex topcoat (latex/latex). The data points are the average of 12 observations, and the bars give the standard deviation.

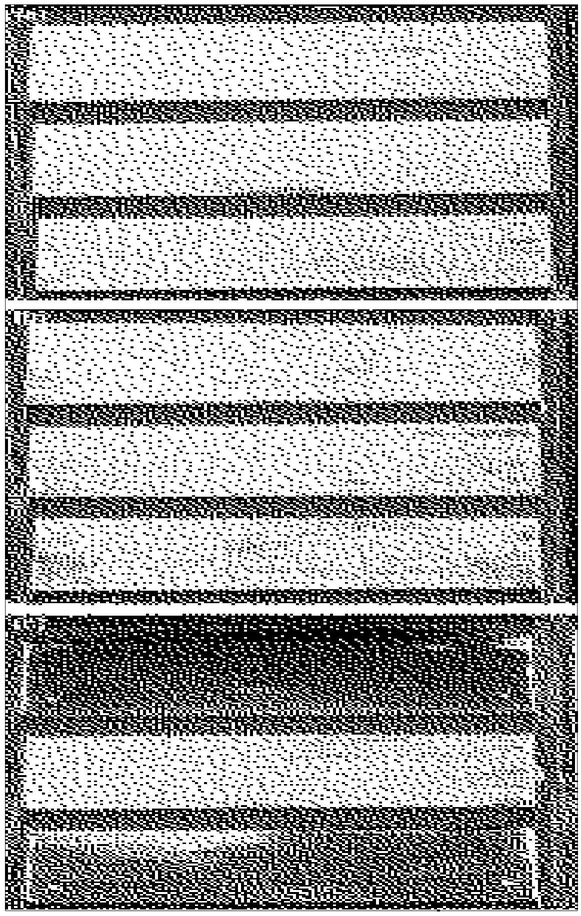

**Figure 12. Example of panels painted with one coat of latex primer and one coat of acrylic latex topcoat (latex/latex) after 17 years of outdoor exposure. (a) control, no exposure prior to painting, (b)preweathered 1 week, and (c) preweathered 16 weeks.**

## 5 ACKNOWLEDGMENTS

We thank Peter Sotos for the years of field evaluations, for maintaining the data of these evaluations, and for the photographs, and we thank John Gangstad for preparing the graphs.

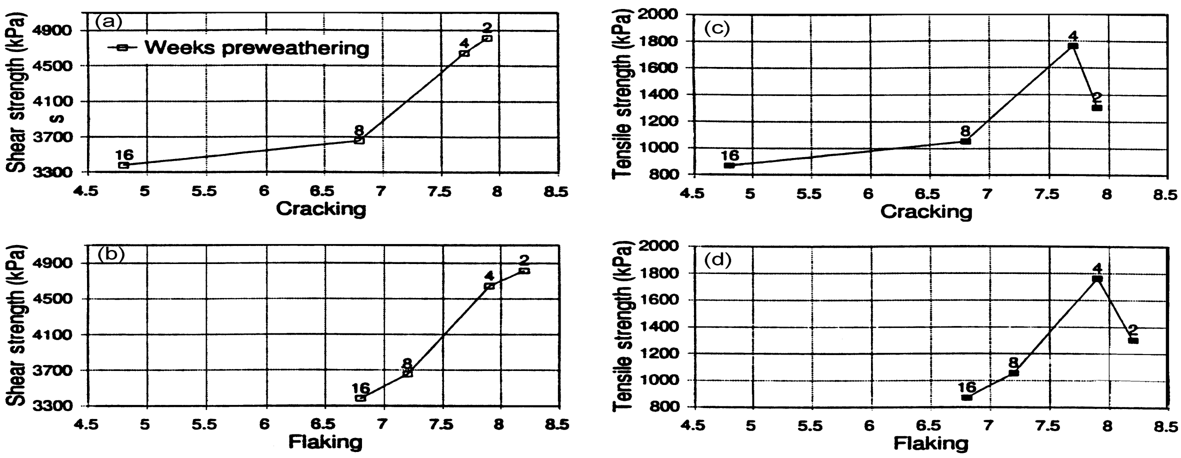


**Figure 13. Paint adhesive strength (shear and tension) of the alkyd-oil primer compared with the cracking or flaking evaluations of the WRP/alkyd/latex paint system after 17 years of outdoor exposure. (a) shear strength versus cracking, (b) shear strength versus flaking, (c) tensile strength versus cracking, and (d) tensile strength versus flaking. The number of weeks of preweathering is shown with each data point. The R2 values are 0.81, 0.98, 0.61, and 0.56 for (a) through (d), respectively. If the 2-week data are not included in the regression analysis for (c) and (d), the R2 values are 0.73 and 0.97, respectively.**

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# Condition Assessment Of 40 Year Old Sewer Stacks In High Rise Buildings

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**Summary: The condition of 40 year old sewer stacks in high rise buildings was evaluated using an electromagnetic non-destructive technique for copper pipes, and invasive cut-out sampling followed by grit blasting and direct physical measurement for cast iron pipes. The samples of approximately 10% of the total population revealed only isolated minor pitting corrosion for both the copper and cast iron stacks, with a negligible probability of failure in the short to medium term.**

**Keywords: Corrosion, graphitisation, condition, assessment, non-destructive evaluation**

## 1 INTRODUCTION

High rise buildings (see Fig. 1 below) contain a vast array of service conduits – electrical, potable water, telephone and sewage to name a few. Electrical and telephone conduits contain cabling, whilst water and sewer conduits transport fluids under the force of gravity or under pumped pressure. The interaction between the conveyant and the conduit material (pipe) may significantly influence the performance and life of the pipe material.

**Figure 1. Multi-storey building constructed in the 1960's**

Following the failure of a small section of sewer pipes in a high rise building which caused major disruption to approximately 100 tenants for several days, it was decided to investigate the condition of sewer stacks in other buildings to determine their likelihood of failure.

### 1.1 Sewer stacks

Sewer stacks transport waste materials from toilets, baths, showers, laundry sinks, washing machines, kitchen sinks and dishwashers into buried sewer mains, and in turn, into the water authorities buried sewer network. Copper and grey cast iron

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were the materials that had been used in the buildings under investigation, with copper being the predominant material. In addition, copper had been used exclusively as the waste pipe in all the buildings. (see Fig. 2).

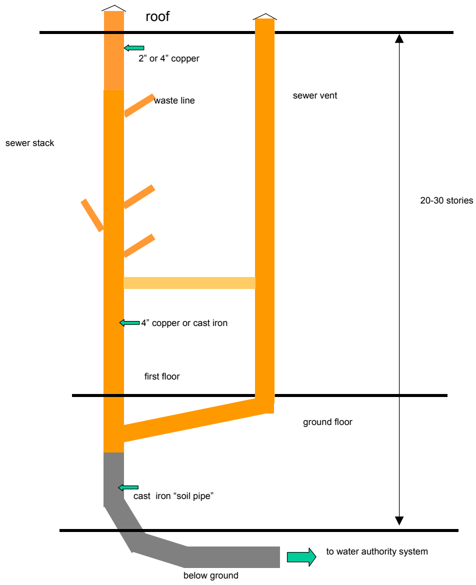


**Figure 2. Schematic of Sewer Stack Configuration**

### 1.2 Corrosion behaviour of copper and cast iron

#### 1.2.1 Corrosion behaviour of copper

Copper is a materiel that has been used extensively in the water industry for well over 40 years and enjoys a reputation for good corrosion resistance, combined with ease of fabrication and widespread availability. Whilst the main use of copper is in reticulating drinking water supplies, it is also used for wastewater disposal.

Corrosion of copper in drinking water is sporadic and tends to occur mainly is soft, weakly buffered waters with low total dissolved solids. In wastewater, failure through pitting or other forms of corrosion is rare when normal conditions prevail. However, the use of strong household disinfectants such as bleach, products containing ammonia and strong acids can cause corrosion of copper water pipes. In some cases, this will increase the soluble copper content of the waste stream to a significant extent as well as causing localised attack of the tube itself.

#### 1.2.2 Corrosion behaviour of cast iron

Cast iron exhibits excellent corrosion resistance when exposed to essentially atmospheric conditions, where water drains away easily. In such an environment the cast iron material undergoes only superficial rusting, and can perform satisfactorily for hundreds of years. If however, copper salts present in solution come into contact with cast iron material, galvanic corrosion can occur, whereby elemental copper plates out on the surface of the cast iron, which in turn undergoes corrosion according to the reaction $Cu^{2+} + Fe \Rightarrow Fe^{2+} + Cu$.

The final corrosion product of cast iron corrosion is a graphite-rich material, which maintains the exact shape and size of the original component, and also affords some degree of structural strength.

## 2 INVESTIGATIVE METHODOLOGY

### 2.1 Condition assessment techniques

Consideration of condition assessment techniques took into account the following:

1. Length of conduits (and relevance of spot measurements);
2. Diameter of conduits (nominally 100mm);
3. Access to surface of conduit (see Fig. 3);
4. Physical properties of copper and cast iron materials;
5. Corrosion behaviour of copper and cast iron materials;
6. Morphology of corrosion of copper and cast iron materials;

7. Need to minimise disruption to services;
8. Cost-effectiveness of technique.

**Figure 3. Access to sewer stack was an important consideration**

Techniques that were considered included:

1. Hand held ultrasonic testing;
2. Hand held low-frequency eddy current “Mainscan”;
3. Cut-out sampling; and
4. Electromagnetic intelligent pigging.

Ultimately it was decided to use the “Saturn” electromagnetic probe supplied by Russell Technologies Incorporated from Canada for inspection of copper stacks (see Fig. 4), and cut-out sampling for inspection of bottom sections of cast iron stacks. The “Saturn” probe enabled the acquisition of essentially continuous wall thickness measurements when lowered down a sewer stack. However, due to the difference in physical properties between copper and cast iron, the same technique could not be used to ascertain the remaining wall thickness of the cast iron stacks.

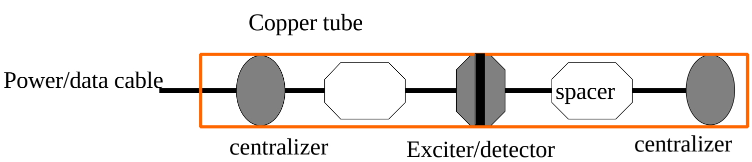


**Figure 4. “Saturn” probe**

Cut-outs was chosen for inspection of cast iron stacks primarily due to:

1. short section of pipe in stacks (approximately 3m);
2. inability of ultrasonics to provide meaningful wall thickness measurements on corroded cast iron;
3. consideration of the most likely corrosion mechanism operating for cast iron pipe; and
4. relatively high cost of procurement of hand-held eddy current device.

Cut-out samples were sectioned longitudinally to reveal internal surface, grit blasted to remove corrosion products, and pitting corrosion and wall thickness measured using micrometer dial gauge and vernier calipers.

## 3 RESULTS

Investigation of copper sewer stacks in 44 buildings revealed a total of 43 defects, all assumed to be attributable to corrosion, and none particularly severe and likely to impact on the life of the stacks in the short term. Figures 5 and 6 show graphical summaries of the results.

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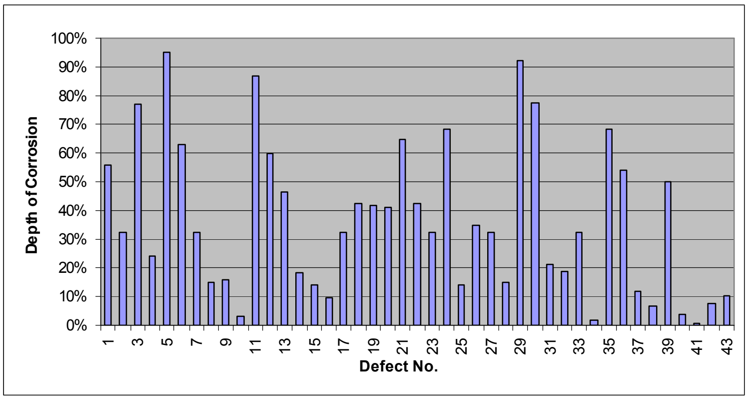


**Figure 5. Depth of Copper Stack Corrosion Defects**

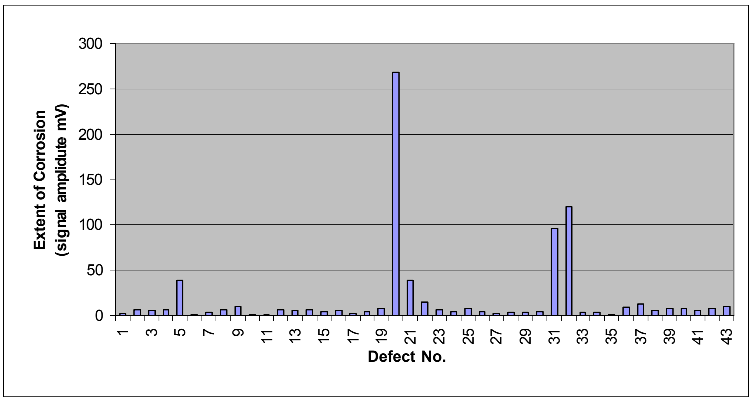


**Figure 6. Extent of Copper Stack Corrosion Defects**

These results show small localised areas of pitting corrosion, with only defect number 21 exhibiting any appreciable loss of volume. However, in combination with only 40% depth of corrosion, does not represent a significant defect.

Similarly, the cast iron sewer stack investigation revealed only minor pitting corrosion on a few samples. The results are tabulated below in Table 1.

**Table 1. Results of Cast Iron Sewer Stack Corrosion**

| Sample code | Minimum Remaining Thickness (mm) | corroded after grit blasting (%) |
|---|---|---|
| **1** | 4.5 | 33% |
| **2** | 2.9 | 44% |
| **3** | 3.8 | 17% |
| **4** | 3.9 | 34% |
| **5** | 3.0 | 46% |
| **6** | 3.5 | 29% |
| **7** | 5.8 | 15% |
| **8** | 2.1 | 77% |
| **9** | 4.7 | 60% |
| **10** | 4.4 | 38% |
| **11** | 4.1 | 25% |
| **12** | 4.6 | 25% |
| **13** | 3.7 | 31% |
| **14** | 4.4 | 40% |
| **15** | 0.0 | 100% |
| **16** | 4.5 | 32% |
| **17** | 2.9 | 74% |
| **18** | 4.2 | 28% |
| **19** | 3.2 | 51% |
| **20** | 3.4 | 49% |
| **21** | 3.8 | 38% |
| **22** | 4.7 | 37% |
| **23** | 3.7 | 34% |
| **24** | 5.1 | 29% |
| **25** | 6.8 | 26% |
| **26** | 3.3 | 46% |
| **27** | 4.4 | 35% |
| **28** | 4.9 | 23% |
| **29** | 5.3 | 35% |
| **30** | 3.2 | 65% |
| **31** | 3.4 | 45% |
| **32** | 3.2 | 41% |
| **33** | 3.5 | 45% |
| **34** | 3.9 | 53% |
| **35** | 3.7 | 38% |
| **36** | 3.2 | 54% |
| **37** | 4.7 | 44% |
| **38** | 4.6 | 42% |
| **39** | 3.5 | 40% |
| **40** | 4.2 | 46% |
| **41** | 2.5 | 51% |
| **42** | 3.2 | 40% |
| **43** | 3.3 | 42% |
| **44** | 1.2 | 90% |
| **45** | 6.1 | 9% |

Samples 8, 15 and 44 exhibit considerable depth of pitting corrosion. However, in the case of samples 8 and 15, the pitting is confined to only a small area, resulting in minimal volume loss, and little reduction in structural strength. There is a greater loss of strength exhibited by sample 44. This can be appreciated by referring to Figs 7, 8 and 9.


**Figure 7. Close up of sample 8 from building 37 showing minor pitting corrosion. Note inspection plate on right hand section.**


**Figure 8. Close up of sample 15 from building 16 showing inspection cover and small through-wall defect on bottom right hand photograph.**


**Figure 9. Close up of sample 44 from building 11 showing pitting corrosion. Also note the eccentric wall thickness.**

Further investigation of sample 44 from building 11 revealed that it is located in a horizontal position conveying water from a laundry. The photograph below (fig. 10) shows the build up of predominantly cellulose material on the wall of the pipe.

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**Figure 10. Close up of sample 44 from building 11 showing build up of waste material.**

## 4 PERFORMANCE OF COPPER AND CAST IRON SEWER STACKS

In general, both materials have exhibited good corrosion resistance when used to convey sewage into the water authorities reticulation system. Very minor pitting corrosion was detected in sparse and isolated locations for the copper tubes, with no evidence of substantial pitting or general corrosion. Only one sample exhibited significant corrosion for the cast iron system, and this appears to be attributable to the accumulation of waste material in a horizontal section of the sewer stack system.

The results of the investigation also indicates that galvanic corrosion, caused by replating of dissolved copper material, is not active, primarily as a result of the good performance of the copper stacks.

Some cast iron stacks exhibit an eccentric bore which causes reduced wall thickness, a consequence of static casting process. When subjected to a corrosive environment and external loading this may lead to reduced life. It is not anticipated that pipes manufactured by centrifugal casting will exhibit eccentric bore.

## 5 CONCLUSIONS

1. Copper stacks investigated have undergone minimal corrosion
2. The vast majority of vertical cast iron stacks investigated have undergone minimal corrosion
3. Some cast iron sewer stacks (soil pipe) have an eccentric bore, which may lead to an earlier failure than would otherwise be realised.

# Estimation Of Residual Service Life For Existing Sewerage Systems

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**Summary: The cost of maintaining public sewer systems in Germany has been estimated at about € 50 billion. According to German environmental laws, the technical state of sewers has to be checked regularly by means of inspection technology. The condition of sewer pipes and pipe joints are evaluated according to the scale and the effects of damage. The determined damages are assigned to one of five different damage classes. The damage classes range from very serious to negligible.**

**In a second stage, the status of sewer sections is evaluated according to the greatest damage. These evaluation data are installed in a sewer database according to belonging functionality and stability variables such as significance of sewer section, hydraulic capacity, overflow frequency, material, construction year, geometry, size of covering and traffic load situation.**

**In a third stage the correlation is graphically described between the network sections and the year of construction and different functionality and stability variables. The aging curves were derived from the available inspection data and the construction year for each status class. The average residual service life of the sewer section is represented by a vertical line between the real age of the sewer section and the point of intersection with the aging curve of intervention status class. The different intersections on the horizontal line with the aging curves of different status classes indicate the ages at which the section is likely to drop to the next class or, going back in time, came from the previous class.**

**The example of a small town shows how the acquisition of data and the evaluation of damaged sewers in a municipality is carried out and illustrates which priorities have to be established during the maintenance of sewer networks. The model city of 8,000 inhabitants is situated in the middle of Germany. The sewer network comprises around 25 kilometers with 700 individual sewer reaches. In 1998 the total sewer system was optically inspected. The results of the inspection serve as the basis for a database.**

**Using inspection results from the model city and the application of the cohort survival model for stock forecasting, a prognosis for the lifetime of sewer pipes has been developed. The first results show that the average lifetime of concrete pipes for drainage of combined waste water can be estimated at 100 years. The average service life of stoneware pipes, both for drainage of dirty waste water and for combined waste water, amounts to 120 to 150 years using these evaluations. Municipal authorities are able to reach decisions regarding inspection and maintenance cycles.**

**Keywords: Sewer Status Assessment, Predictive Rehabilitation Planning, Aging Process, Forecasting, Service Life**

## 1 INTRODUCTION

The orderly maintenance of sewer systems requires systematic and regular optical inspection and documentation of the results. Generally, the inspection is conducted by remote control CCTV camera systems, inspection experts, and, in some cases, a leak test. The inspection of the total sewer network district usually must be carried out every ten years. In order to document the results, video recordings, photographs, test certificates and sketches have to be produced (Buetow *et al.* 1995).

As a result of the inspections, local councils can decide themselves upon subsequent construction or repair measures. The urgency of the necessary measures is determined by the danger to environment, functionality and stability. According to an investigation from 1997, about 18 percent of the German public sewer network, which consists of different pipe materials (concrete, stoneware and plastic) and has a total length of about 440,000 kilometres, is considerably damaged due to its partially high age (Kaempfer e*t al.* 1999).

The rehabilitation of these sewers would cost about € 50 billion. At the present time only 3.5 percent of this is available. If one considers that each year the aging process causes at least one additional percentage point of network length to become urgently in need of rehabilitation, then one can easily realize that significant improvement of sewer conditions cannot be achieved. Therefore an effective improvement of the condition of sewer networks with the well-known limited financial resources necessarily requires early recognition and timely and thus inexpensive removal of incipient defects (Kaempfer *et al.* 2000).

It is accepted that, due to sewer damages, about 500 million cubic meters of contaminated liquids leak into soil and ground water every year (Buetow *et al.* 1995). Therefore, the main goal of rehabilitation planning should be to predict sewer deterioration in advance and thus avoid gradual loss of intrinsic value. Long term maintenance at minimal cost must take into account the optimal residual service life of the sewer network system. This requires early recognition of incipient damages.

## 2 STRUCTURE OF EVALUATION MODEL

The example of the model city Stadtilm shows how the acquisition of data and the evaluation of damaged sections in a municipality is carried out, and illustrates precisely which priorities have to be established during the repair of sewers. This model city is a small town of 8,000 inhabitants in the middle of Germany. The sewer network comprises around 25 kilometers with about 700 sewer sections, primarily for the drainage of dirty waste water (8 %), rainwater (6 %) and 86 % combined sewers (Kaempfer *et al.* 2000).

The total sewer system was inspected two years ago. The age and structural materials are typical for the historical development of the infrastructure of small towns in the middle of Germany built at the turn of the 19th century. The results of the inspection serve as the basis for the repair of the sewer network, for which an annual budget of only € 150.000 is available. A comprehensive data base, incorporating the results of the inspection, was established.

The evaluation model was developed as a part of the project “Residual Service Life for Sewerage Systems”, at the Institute for Material Research and Testing at the Bauhaus University Weimar (Kaempfer *et al.* 2000). In order for such an evaluation model to be used by a municipality, it must be relatively simply constructed and has to establish clear priorities that can be put into practice. The evaluation model can be divided into three stages of development. First, the individual instances of damages are evaluated, then the influence variables with regard to their effects of damage on environment, functionality, and stability are estimated, and finally an overall evaluation of the situation is made.

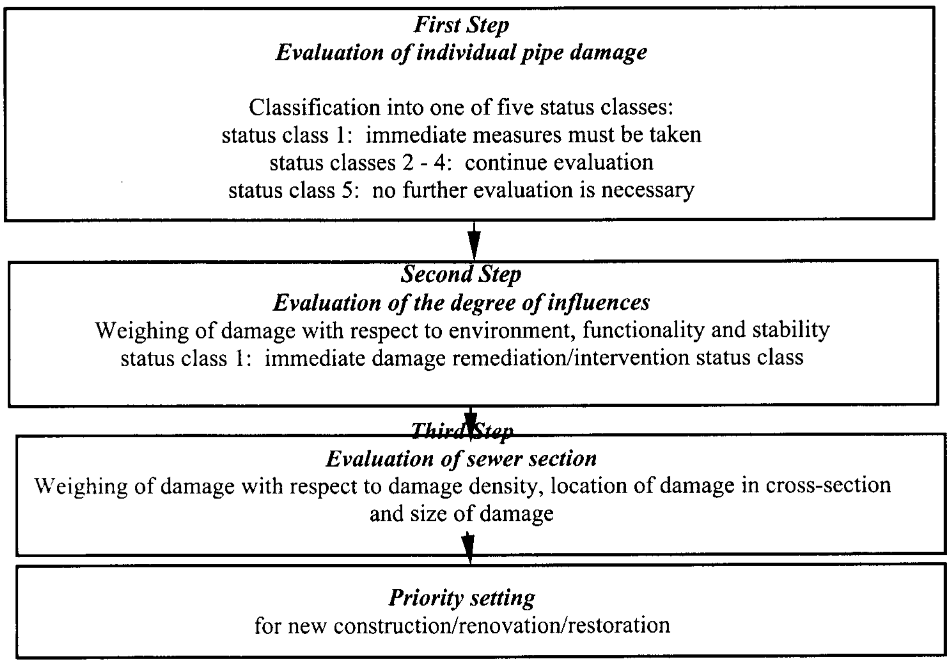


**Figure 1. Diagram of the evaluation model for damaged sewer sections**

Each instance of damage is assigned to one of five different status classes. This classification requires the knowledge of the exact geometrical dimensions of the damage, such as width and depth of cracks, fractures, and defective pipe connections.

The basis of the classification of sewer section status is its most recent machine-readable inspection report, and for each problem detected it contains the type and extent of damage, as well as the location in cross-section and in direction of flow. Each instance of damage is assigned to one of five damage classes from status class SC 1 - very serious damage- to status class SC 5 - negligible damage. During evaluation of the individual instance of damage, the effects on the environment, functionality and stability are assessed.

The classification into different status classes is made on the basis of a damage catalogue (Kaempfer *et al.* 2000). The rehabilitation priority for the entire sewer is determined by the level of damage of each section. The determining level is thus the level of the most serious damage. If the rehabilitation priority drops to the intervention class, the section is in urgent need of rehabilitation. The intervention class can also depend on location. The rehabilitation priority bears no relation to the remaining lifetime of the sewer section. The forecast of rehabilitation priorities uses a model of the local aging process or is determined by inspection findings and the construction years of all the sections (Hochstrate 2000).

The aging process of sewers varies from place to place with regard to average service life. It is nevertheless evident that, for sewer sections, the past speed of aging will be maintained in the medium term future. Repeated inspections offer the opportunity to recognize changes in the aging speed and to generate forecasts that take this information into account.

The modeling of the aging process of sewer sections is based on the fully developed statistics to compute human survival probabilities. Sewer sections and human beings have similar life expectancies, but a completely different distribution of age at death. Many sewers fail between the age of 0 and 10 years because of faulty installation and construction defects or overloads. On the other hand, some sewer sections, which have already attained an age of 100 years in good condition, have a higher survival probability than a new one that has recently been laid.

In order to evaluate the expected residual lifetime of sewer sections it is important to consider the individual status classes. The range of status values is regarded as a constant against which a continuous function of age is plotted. Figures 2 and 3 show examples of the change-of-condition functions of different pipe materials for the five status classes. They can be used to estimate the medium service life of a sewer section or to ascertain the amount of time it will spend in one of the five status classes. Each sewer system must have its own set of change-of-condition functions, according to its historical infrastructure development, based on the inspection data base.

The change-of-condition is updated as new inspection data becomes available. The conditions for estimation of residual service life for existing sewer sections are the knowledge of both the construction and inspection years and the availability of status class values for rehabilitation priority. On the basis of the normatively established intervention status class it is possible to forecast the repair year and the year of complete rehabilitation. Forecasts of time of intervention are obtained by the point of intersection of the vertical line (year of construction) with the aging curve of status class one.

## 3 FORECASTS OF SEWER STATE

The evaluation model has been installed in a central sewer system database. All data was entered into the sewer system database that allowed a comprehensive evaluation of damage to the sewer system with respect to stability, functionality, and environment. Besides damage data, a number of other essential data were recorded, such as material and construction parameter, year of construction, covering depth, and traffic loads. All damages were allocated to the corresponding damage classes.

Most damages in Stadtilm are allocated to status class 3 and 4. The number of cases of damage that needed to be dealt with immediately was surprisingly high. As this large number of cases cannot be remedied at once, it must be decided which rehabilitation measures are most urgently necessary. The location of the worst cases of damage in the city is of essential importance for planning maintenance work and infrastructure maintenance planning generally. The sewer database was linked to a GIS system.

The modelling of the aging process of sewer networks is based on change-of-condition functions. These functions were derived from the available inspection data and the construction years. The vertical axis in diagrams 2 and 3 shows the proportion of construction years.

The horizontal axis represents the sewer section age in years. From these diagrams, the average service lifetime of sections of definitive age can be obtained. The residual lifetime can be determined by the period between data for the vertical line at the age of x years for a defined status class of the sewer section and the point of intersection of the horizontal line with the defined intervention status class (urgent rehabilitation priority SC 1). Figure 2 gives an example of the network transition function for the Stadtilm project. The curves were derived from the inspection data of the whole sewer network with construction years.

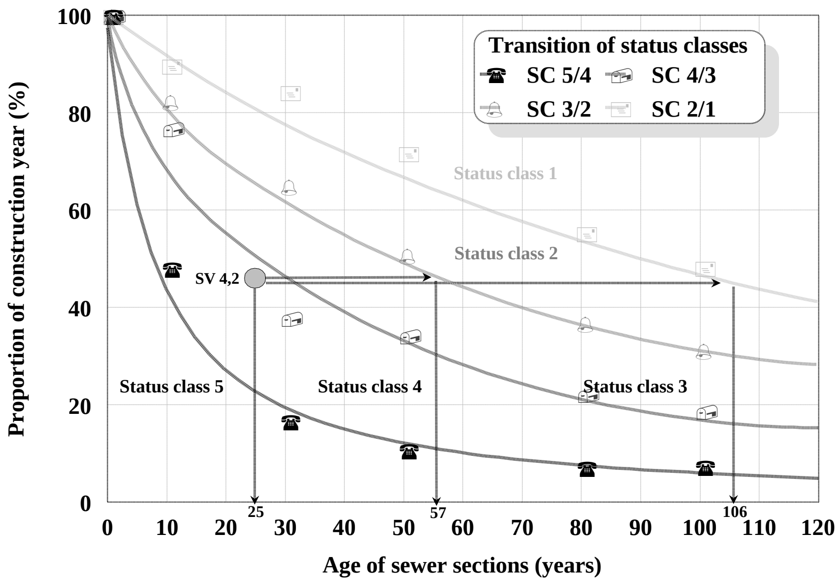


**Figure 2. Status transition functions for local aging process of concrete sewer sections**

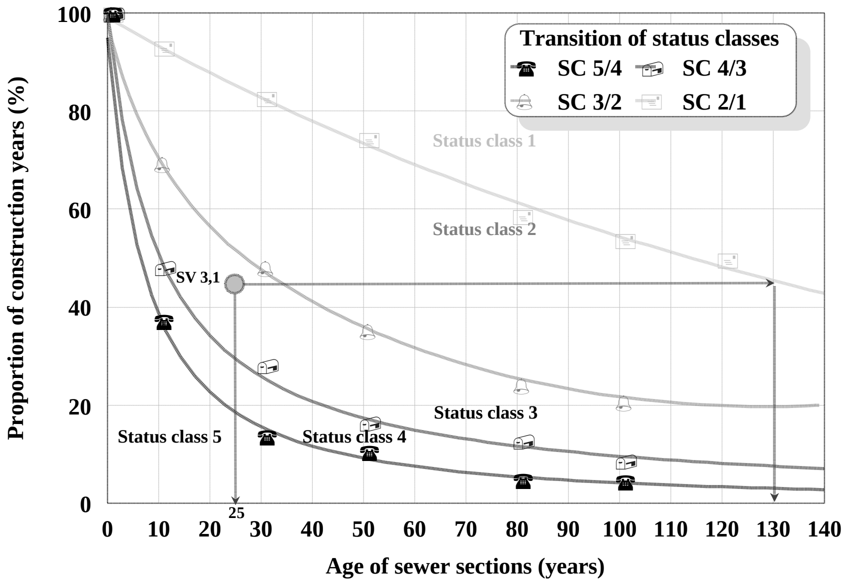


**Figure 3. Status transition functions for local aging process of stoneware sewer sections**

Concrete pipe sections, which are today 25 years old, will serve as an example (see Figure 2). On the line of inspection age 25 interpolate the status value SV 4,2 between the points for status values 4 and 5. Draw the horizontal SC-path. Transition to class SC 2 occurs at age 57 and to status class SC 1 at age 106.

The average residual service life of concrete pipe sewer sections is accordingly, on average, 32 years in water preserve area and 81 years in normal waste water situation. Figure 3 shows the change-of-condition function for the local aging process of stoneware sewer sections. The change-of-condition functions describe the average statistical service life of sewer pipe materials in one status class.

The presented regression lines are determined by statistical calculus of observation. The statistically-determined values amount from 60 to 94 percent. The determined aging curves apply only to the drainage district. In other cases the curves will assume other behaviour.

Contrary to the results so far the regression lines of aging show a pronounced degressive development. The sojourn time increases in decreasing status classes. The average service life of concrete sewer sections amounts 100 years. In the case of drinking water safety areas, the effective time period of repair is 58 years. Compared with concrete pipe sewers the aging process of stoneware sewer sections shows a more degressive course. Stoneware is a more brittle material, a characteristic which can result in sudden material failures.

The average retention time of stoneware sewers in the status classes from SC 5 to SC 3 is lower in comparison to concrete sewers. On the other hand, the sojourn time to achieve the intervention status class SC 1 is about 50 percent higher than that of concrete pipe sewers. The total residual service life of stoneware sewer sections ranges from 120 to 150 years. Especially during or directly after installation of stoneware pipes, a high amount of sewer damages arises (Kaempfer *et al.* 2000).

The estimation of residual service life of sewer sections forms the basis for advanced planning of sewer inspection, the representation of the sewer section condition expected in the future and for the yearly budgeting of rehabilitation measures.

## 4 REAL AND ARTIFICAL AGING OF PIPE MATERIALS

Manufacturing methods and pipe material parameters depend on construction years. From 158 building sites, different kind of pipe materials were removed to determine material parameters for the natural aging process. The investigated pipe materials were installed between 1900 to 1985. The vast majority of pipes, installed during this period were stoneware and concrete pipes. The material parameters for concrete pipes, that were primarily tested, were comprehensive strength, porosity and water absorption. The tensile bending strength, the porosity and structure failures were analysed on stoneware pipes. Table 1 shows, for example, average material parameters of different naturally-aged concrete pipes from combined sewers.

**Table 1. Material parameters of real aged concrete pipes from combined sewers**

| *Age of samples (years)* | *Compressive strength (N/mm²)* | | *Porosity (%)* | *Water absorption (%)* |
|---|---|---|---|---|
| | *at spring line* | *at the crown* | | |
| 70 | 34 | 28 | 16,2 | 1,98 |
| 40 | 32 | 25 | 11,1 | 1,70 |
| 25 | 42 | 32 | 16,7 | 2,51 |
| 15 | 52 | 45 | 16,5 | 3,13 |

The average compressive strength of analyzed naturally-aged concrete samples is beyond all exceptions. From the damage grade of concrete pipe material it is possible to determine the residual load-carrying capacity. Even in reduction of pipe wall thickness until 60 % of original wall thickness the residual load-carrying capacity may be sufficient.

To simulate resistance to biogenous sulfuric acid attack, a very simple test procedure has been developed that provides an authentic demonstration of how sulfuric acid attacks in dirty and combined sewers. In these tests waterlogged specimens are weighed and stored in plastic containers. Each container is filled with diluted sulfuric acid (pH 2). The pH value of the acid bath is kept constant over 70 days. This test procedure is suitable for decision on service life under time-accelerated conditions.

All six days the specimens are removed from the solution, then stored under running water, vigorously brushed off and all loose parts removed. After this the specimens are weighed and stored, following the same procedure. This operation corresponds to practical conditions of cyclic flooding in combined waste water sewers after downpours. The acid is circulated permanently. In order to keep the pH value constant, concentrated acid is added. The acid bath is restored after each storage cycle (Kaempfer *et al.* 1999).

Several types of stoneware and concrete pipe materials of very different ages were investigated. The undamaged cross-section, the residual weight, the residual compressive and tensile bending strength and the acid penetration depth at different exposition times in diluted sulfuric acid were determined, as to be able to draw conclusions about the further aging process of older pipe materials. Figure 4 show the development of weight losses of time-accelerated aged concrete pipe material in diluted sulfuric acid.

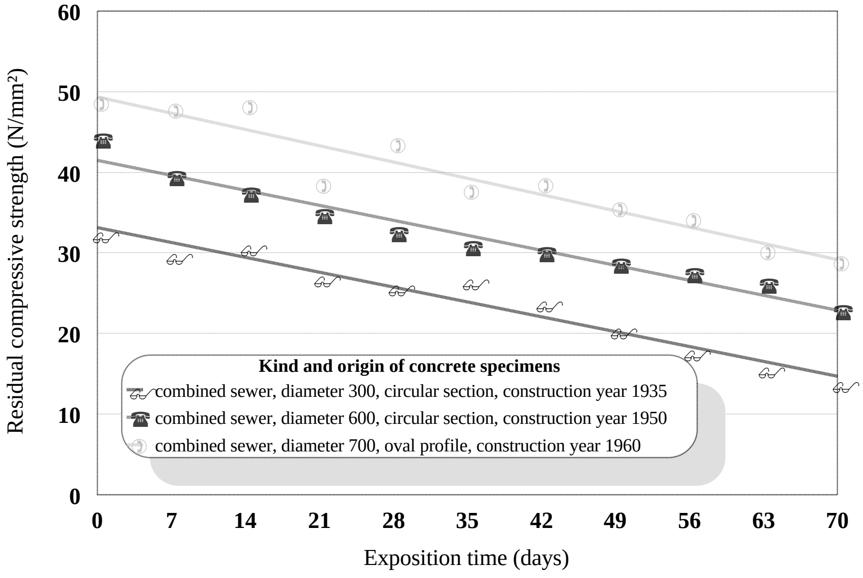


**Figure 4. Residual compressive strength of concrete pipe materials versus exposition time in diluted sulfuric acid**

## 5 SUMMARY

The forecasts of the aging process for different pipe materials make use of a model of the local aging process based on inspection findings, pipe material parameters determined by naturally-aged pipe materials, time-accelerated laboratory tests and construction years of all the sections. The aging process of sewers varies from place to place with regard to mean length of use and relative sojourn times in status classes.

Local aging processes of sewer sections cannot even be causally explained, so diverse are the interaction between types of construction, manner of installation and the uses that are made of them. Thus it can be assumed, that a section's past speed of aging will also be maintained in the medium term future.

Determination of parameters on naturally-aged pipe materials and time-accelerated laboratory tests offer the opportunity to recognize changes in aging speed and to generate forecasts that take this information into account.

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# The Effect Of Coatings On The Service Life Of Concrete Facades

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**Summary: The effect of coatings on the service life of concrete facades was studied by computer simulation and tests. Computer simulation makes it possible to study the behaviour of a structure as exposed to normal outdoor weather. The effect of coatings on the moisture content, frost damage, carbonation and corrosion of reinforcement in a structural cross-section was examined.**

**The impact of a coating on the service life of a concrete structure depends on both the original permeability properties and the rate of deterioration of the coating. For durability design a special coating factor of service life was defined both with respect to frost damage and carbonation. The coating factor expresses the relative extention of service life of a coated structure as compared to the service life of an uncoated structure. By some impregnation agents and organic coatings the coating factor with respect to frost damage may be as high as 6-8. With respect to carbonation organic coatings seem to be most effective. The coating factor by some organic coatings may be as high as 10.**

**Keywords: Service life, coating, concrete facade.**

## 1 INTRODUCTION

Both the aesthetic appearance and durability of concrete facades may be enhanced by coatings. Even if the main purpose of the coating would be aesthetic the effects on durability must also be considered as they may not always be positive.

Coatings act like barriers against moisture and aerial gases. They retard the flow of liquid moisture and gases through the surface of concrete in both directions. Especially the following phenomena depend on the permeability properties of coatings:

- capillary uptake of rain water into concrete,
- evaporation of pore water from concrete to outside air, and
- diffusion of aerial gases, especially $CO_2$, into concrete.

All degradation factors of concrete structures depend on the moisture content of concrete either directly or indirectly. So the influence of coatings on the moisture content is extremely important when evaluating their effects on the durability of concrete and reinforcement. Usually coatings retard both the wetting and drying processes. The total impact in varying weather conditions is often difficult to predict. In a facade element of a building there is also a moisture flow from the inside of the wall to the outside that has to be considered. The problem becomes still more complicated by the deterioration of coatings themselves because the permeability properties are changed by deterioration. So the final evaluation of coatings is not an easy task and is only possible by either extensive experience or sophisticated moisture physical calculations combined with material tests.

The moisture content of concrete has a direct influence on the internal frost damage of concrete. If the moisture content of concrete can be preserved below the critical content no frost damage is possible. So by preventing or retarding the water uptake into concrete the coatings presumably improve the durability of concrete with respect to frost damage. However, the protective properties of a coating may degrade in the long term when the permeability of the coating is increased by deterioration. Completely impermeable coatings can hardly be used in facades as there is normally a thermal moisture flow through the wall. Also an impermeable coating may prove to be very vulnerable in long term as small defects in the coating may cause a great local increase in the moisture content endangering adhesion of the coating and frost resistance of concrete.

The impact of coatings on carbonation of concrete is both direct and indirect. On one hand the carbonation is retarded as the coating forms a physical barrier to the diffusion of carbon dioxide into concrete. On the other hand the rate of carbonation is accelerated by coatings that keep the structure dryer as compared to the moisture content without coating. That is because the flow of $CO_2$ through open concrete pores is more rapid than the flow through water filled pores. The total effect is again difficult to predict.

The corrosion of reinforcement is highly dependent on the moisture content of concrete. The rate of corrosion decreases by low and very high moisture contents. The maximum corrosion rate is found to be around 95% RH.

The degradation of concrete facades is a complex function of weather conditions, properties of concrete, structural details, protective impact of other structures and properties of coatings. There are also several degradation processes and these may interact. For instance frost damage may promote both the carbonation and corrosion of reinforcement. These complicated processes can hardly be studied by direct exposure tests. However, they can be studied by computer simulation combined with laboratory tests.

# 2 COMPUTER SIMULATION AND RECOMMENDATIONS OF SERVICE LIFE DESIGN

## 2.1 General

The effects of coatings on the degradation of concrete facades was studied with the aid of a computer simulation programme by which the temperature, moisture content, frost damage, carbonation and corrosion of reinforcement can be reproduced in a structural cross section. The structure was exposed to simulated weather from outside. The weather models consider both daily and seasonal changes of temperature, relative humidity, rain, wind and solar radiation. All the degradation processes are studied simultaneously which enables consideration of interaction of degradation factors. The increment of time in the step by step calculation process is one hour, which is small enough to consider also the hourly variation of weather stresses (Vesikari 1999a and b).

Design recommendations were given for coating protection against frost damage and carbonation using the factor approach (Vesikari 2000). A special coating factor of service life was defined both with respect to frost damage and carbonation. The coating factor expresses the relative extension of service life of a coated facade as compared to that of an uncoated facade:

$$t_{Lc} = A_c \cdot t_{Lu} \tag{1}$$

where

$t_{Lc}$ is service life of a structure with coating,

$t_{Lu}$ service life of a structure without coating, and

$A_c$ coating factor of service life.

The designer can choose the recoating frequency but it affects the coating factor of service life both with respect to frost damage and carbonation.

## 2.2 Moisture content and frost resistance

### 2.2.1 Computer simulation of moisture content

Well known moisture physical calculation methods were used in the computer simulation of moisture content. The rate of water uptake was controlled by the capillary index (k) and the rate of evaporation from concrete was controlled by the moisture transfer coefficient (β).

By doing capillary water uptake tests in a laboratory with coated and uncoated concrete specimens the reduction factor for the capillary index was defined and determined as follows:

$$m_{k0} = \frac{k_{c0}}{k_u} \tag{2}$$

where

$m_{k0}$ is reduction factor of the capillary index (new coating),

$k_{c0}$ capillary index of coated concrete (new coating), kg/(m$^2\sqrt{}$s),

$k_u$ capillary index of uncoated concrete, kg/(m$^2\sqrt{}$s).

Correspondingly by doing drying tests with coated and uncoated concrete specimens the reduction factor for the moisture transfer coefficient was determined as follows:

$$m_{e0} = \frac{b_{c0}}{b_u} \tag{3}$$

where

$m_{e0}$ is reduction factor of the moisture transfer coefficient (new coating),

$\beta_{c0}$ moisture transfer coefficient with coated concrete specimen (new coating), kg/(m$^2$sPa),

$\beta_u$ moisture transfer coefficient with uncoated concrete specimen, kg/(m$^2$sPa).

The rate of deterioration in coatings was studied in a laboratory by aging tests. Concrete specimens with coatings were exposed to both UV radiation and freeze-thaw cycles. After the period of aging exposure the water uptake test was repeated. The deterioration of coatings was defined by the increase of the reduction factor of capillary index as follows:

$$m_k = \sqrt{m_{k0}^{\ 2} + f(t) \cdot (1 - m_{k0}^{\ 2})} \qquad (4)$$

where

$m_k$ is reduction factor of the capillary index with a deteriorated coating,
f(t) deterioration function (0 < f < 1),

t time, years.

Correspondingly the reduction factor of the moisture transfer coefficient was determined as follows:

$$m_e = m_{e0} + f(t) \cdot (1 - m_{e0}) \qquad (5)$$

where
$m_e$ is reduction factor of the moisture transfer coefficient with a deteriorated coating.

Assuming a linear degradation rate we can write:

$$f(t) = v_f \cdot t \qquad (6)$$

where

$v_f$ is rate of deterioration of coating, 1/s.

The relationship between the rate of degradation in natural conditions and that in the laboratory exposure test was roughly evaluated by experience.

The moisture content of concrete varies depending on the rate of wetting and drying at each moment. The moisture content varies at hourly, daily and monthly levels. If the structure is sheltered from rain the moisture content varies only a little with the varying relative humidity of air. Normally the relative humidity alone is not able to raise the moisture content high enough to endanger the adhesion of coatings or the frost resistance of concrete.

When the structure is exposed to rain the moisture content of concrete normally stays low as long as the coating has not deteriorated. By increasing deterioration of the coating the moisture content of the concrete rises to approach the critical limit both with respect to adhesion failure of the coating and frost damage of concrete. If frost damage takes place the coating completely loses its protective properties. However, even if no frost damage would occur the service life of the coating is considered to end at a high moisture content because the solar radiation causes high cycling pressures on the coating, normally resulting in an adhesion failure. The critical moisture content with respect to adhesion loss is close to that with respect to frost damage, both representing a state when all the capillary pores are filled with water.

### 2.2.2 Evaluation of the service life of coating by the critical moisture content

The moisture content of concrete surfaces exposed to rain was studied by computer simulation. The values of the reduction factors $m_k$ and $m_e$ were varied between 0 and 1. From the results the limiting values for $m_k$ by which the critical moisture content is never exceeded were determined as a function of $m_e$. The results could be expressed mathematically in the following form:

Vertical surface: $$m_k \leq 0.6 \cdot m_e^{\ 0.6} \qquad (7)$$

Horizontal surface: $$m_k \leq 0.35 \cdot m_e^{\ 0.65} \qquad (8)$$

The critical lines of $m_k$ as a function of $m_e$ are presented graphically in Figure 1.

Coatings that do not fulfil the requirements of Equations 7 and 8 are not recommended in use on exposed concrete facades as the service life of them is probably short. Usually the facade coatings meet this requirement when new but as a result of deterioration they later lose their protective property.

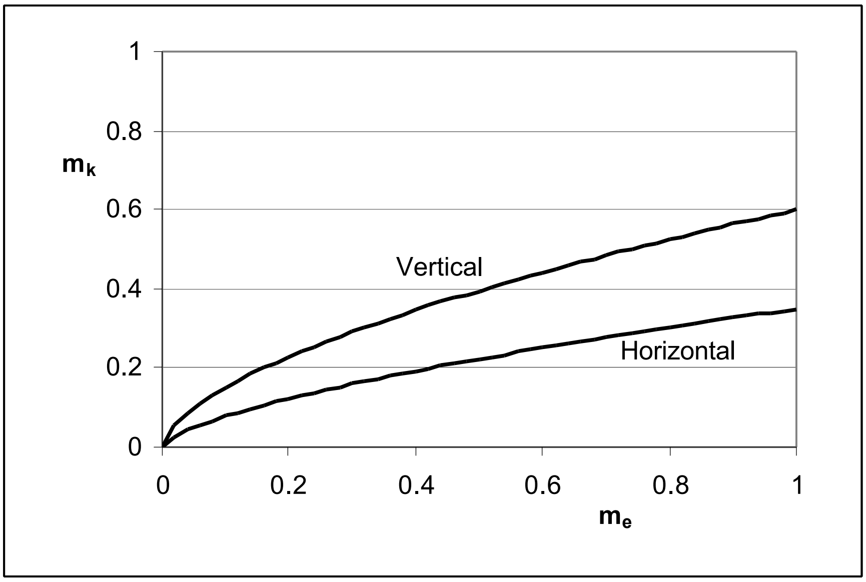


**Fig 1. The critical lines below which the moisture content of concrete is always low enough to prevent adhesion failure and frost damage on vertical and horizontal exposed surfaces.**

The service life of a coating can be evaluated by inserting terms from Equations 4, 5 and 6 to the condition of Equation 7 on a vertical surface and the condition of Equation 8 on a horizontal surface and solving for the time t.

### 2.2.3 Rules for service life design

In the service life design of coatings the service life of coating is evaluated conservatively by assuming that the deterioration of a coating only effects the water uptake and not the drying process. Then we insert only Equations 4 and 6 to the conditions of Equations 7 and 8 and we get:

Vertical surface
$$t_{Lc} = \frac{0.36 \cdot m_{e0}^{1.2} - m_{k0}^{2}}{v_f \cdot (1 - m_{k0}^{2})} \tag{9}$$

Horizontal surface
$$t_{Lc} = \frac{0.12 \cdot m_{e0}^{1.3} - m_{k0}^{2}}{v_f \cdot (1 - m_{k0}^{2})} \tag{10}$$

where

$t_{Lc}$ is the service life of coating, years.

The coating safety factor is determined from the formula:

$$g_{tc} = \frac{t_{Lc}}{t_{Rc}} \tag{11}$$

where

$\gamma_{tc}$ is safety factor of the coating and

$t_{Rc}$ period of recoating (given by the designer), years.

The coating factor of service life with respect to frost resistance of the structure, $A_{cf}$ (ref. Formula 1), depends on the coating safety factor $\gamma_{tc}$. The factor $A_{cf}$ increases by increasing values of $\gamma_{tc}$ as presented in Table 1. The values apply to both vertical and horizontal surfaces.

**Table 1. Coating factor of service life with respect to frost damage, $A_{cf}$.**

| Coating safety factor, $\gamma_{tc}$ | 1 | 1.5 | 2.0 | 2.5 | 3.0 |
|---|---|---|---|---|---|
| Coating factor of service life, $A_{cf}$ | 1.5 | 4.5 | 6.9 | 8.3 | 9.1 |

Several coating systems for concrete facades were tested in a laboratory. The quantities $m_{k0}$, $m_{e0}$ and $v_f$ were evaluated for each coating based on tests. The service life and the safety factor of each coating system was determined as presented above and the coating factor for service life was determined from Table 1.

By some impregnation agents and organic coating systems reasonable service lives for coatings ($t_{Lcoating}$ > 10 - 30 years) were obtained on vertical surfaces. The coefficient of service life with respect to frost damage could be as high as 6-8. The cement based coatings did not prove to be effective in the protection from frost damage.

Very few coating systems can be recommended on exposed horizontal surfaces. For many coatings the calculated service life appeared to be negative, indicating that these coatings may accelerate the rate of frost damage in concrete. However, some impregnation agents and organic coatings may be beneficial.

## 2.3 Carbonation

### 2.3.1 Computer simulation of carbonation

The progress of carbonation was determined stepwise by computer simulation. For each time increment the corresponding increment of carbonation depth was determined using relevant parameter values. The parameters of carbonation were the temperature and relative humidity of concrete, depth of carbonation (at the beginning of the time increment), coating parameters, etc. Mathematically the increment of carbonation depth on an uncoated surface can be expressed as follows:

$$\Delta x_{0i} = \frac{k_{ca;i}^2}{2 \cdot x_i} \cdot \Delta t_i \qquad (12)$$

where

$\Delta x_{0i}$ is increment of carbonation depth of concrete without coating within the i^th time increment, m,

$k_{ca;i}$ coefficient of carbonation of concrete at i^th time increment (depending on moisture content of concrete), $m/s^{0.5}$

xi carbonation depth of concrete without coating at the i^th time increment, and

$\Delta t_i$ i^th time increment, s.

The increment of carbonation depth of concrete with a non-deteriorated coating is determined as follows:

$$\Delta x_{1i} = \Delta x_i \cdot m_{ca;i} \qquad (13)$$

where

$\Delta x_{1i}$ is increment of carbonation depth of concrete with a non-deteriorated coating within the i^th time increment, m,

$\Delta x_i$ increment of carbonation depth of concrete without coating within the i^th time increment, m (ref. Formula 12) and

$m_{ca;i}$ the reduction factor of carbonation depth at the i^th time increment.

The reduction factor of carbonation depth is determined from the following formula:

$$m_{ca;i} = \frac{\sqrt{1 + 2\dfrac{R_b}{x_{1;i-1}}}}{1 + \dfrac{R_b}{x_{1;i-1}}} \qquad (14)$$

where

$R_b$ is the equivalent concrete thickness of the coating with respect to diffusion of carbon dioxide, m.

The value of $R_b$ is determined by simple carbonation tests with coated and uncoated concrete specimens.
As the value of $x_1$ is still unknown, the value of factor $m_{ca}$ is determined using the known value of $x_1$ at the earlier time interval. If the increment of time is one hour the error is insignificantly small.

The increment of carbonation depth of concrete with a deteriorated coating is determined as follows:

$$\Delta x_{2i} = \Delta x_i \cdot (m_{ca;i} + f_i \cdot (1 - m_{ca;i})) \qquad (15)$$

where

$\Delta x_{2i}$ is the increment of carbonation depth of concrete with a deteriorated coating within the i^th time increment, m and

$f_i$ value of the deterioration function at the i^th time increment.

In the case of constant degradation rate the value of the deterioration function is determined as follows:

$$f_i = v_f \cdot t_i \qquad (16)$$

where

$t_i$ is total time in the beginning of the $i^{th}$ time increment.

The total carbonation depths $x_0$, $x_1$ and $x_2$ are then determined as follows:

$$x_{ni} = \sum_i \Delta x_{ni} \qquad n = 0,1 or\, 2 \tag{17}$$

where

$x_{ni}$ is carbonation depth on concrete after the $i^{th}$ time increment, m, and

$\Delta x_{ni}$ increment of carbonation depth during the $i^{th}$ time increment, m.

In practice all the three carbonation depths, $x_0$, $x_1$ and $x_2$, are determined simultaneously. As an example the carbonation depths have been determined on a concrete facade as sheltered from rain in Figure 2. In Figure 3 the carbonation depths have been determined when exposed to rain on a vertical concrete facade.

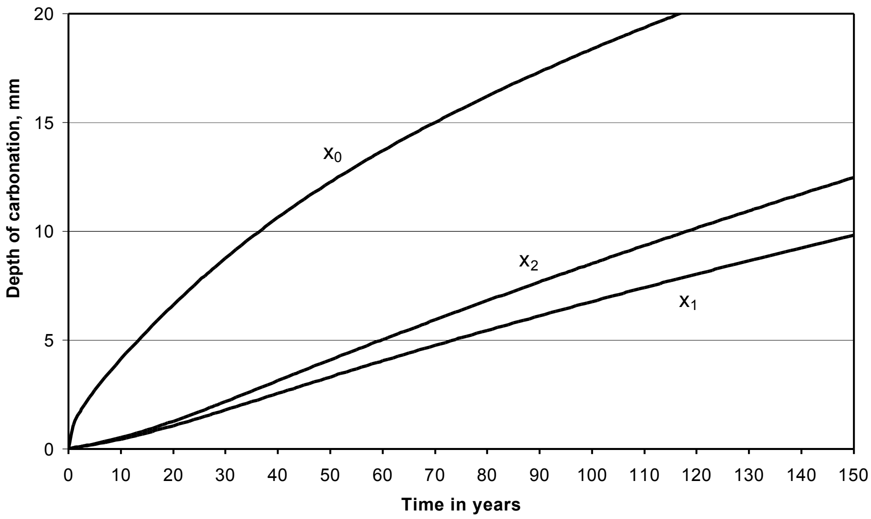


**Fig 2. Progress of carbonation depth on a concrete facade as sheltered from rain:**

**$x_0$ at holes of the coating, $x_0$,**
**$x_1$ under a non-deteriorated coating, and**
**$x_2$ under a deteriorated coating on an average.**

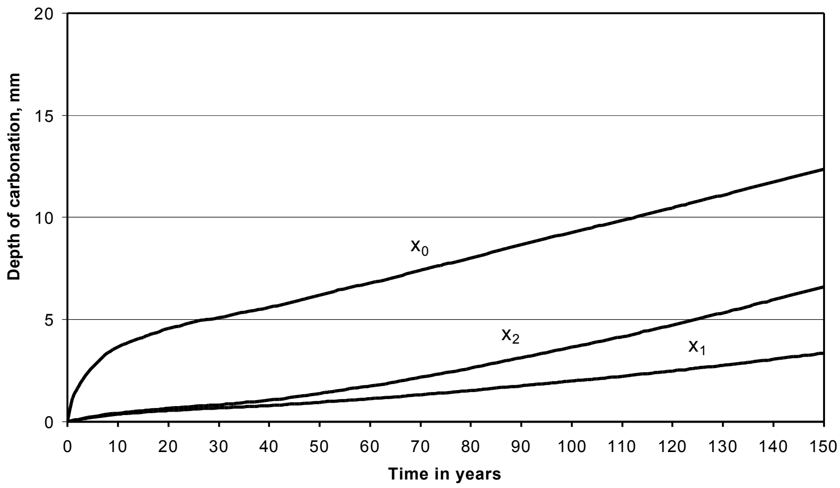


**Fig. 3. Progress of carbonation on an exposed vertical concrete facade:**

**$x_0$ at holes of the coating, $x_0$,**
**$x_1$ under a non-deteriorated coating, and**
**$x_2$ under a deteriorated coating, as an average**

### 2.3.2 Rules for service life design

The coating factor of service life with respect to carbonation (ref. Formula 1), can be presented in the following form:

$$A_{cc} = (1 + \frac{2 \cdot R_b}{c_c} a_f) \quad (18)$$

where

$A_{cc}$ is coating factor of service life with respect to carbonation,

$c_c$ concrete cover, m and

$a_f$ deterioration factor of coating with respect to carbonation.

Without any deterioration the deterioration factor of the coating is 1. If deterioration takes place an approximate function was derived for $a_f$. The parameters of this approximate function were $R_b/c_c$ and f. The value of the deterioration function f is determined at the end of the recoating period:

$$f = v_f \cdot t_{Rc} \quad (19)$$

The coating factors of service life with respect to carbonation can be read from Table 2 as a function of parameters f and $R_b/c_c$. The recoating period is chosen by the designer.

**Table 2. Coating factors of service life with respect to carbonation.**

| Deterioration function after the recoating period f | Coating factor $A_{cc}$ $\frac{R_b}{c_c}$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 0.1 | 0.25 | 0.5 | 1 | 2 | 4 | 8 | 16 | 32 |
| 0.00 | 1.20 | 1.50 | 2.00 | 3.0 | 5.0 | 9.0 | 17.0 | 33.0 | 65.0 |
| 0.05 | 1.20 | 1.50 | 1.98 | 2.9 | 4.8 | 8.3 | 14.9 | 26.9 | 48.3 |
| 0.10 | 1.20 | 1.49 | 1.97 | 2.9 | 4.6 | 7.7 | 13.2 | 22.7 | 38.5 |
| 0.15 | 1.20 | 1.49 | 1.95 | 2.8 | 4.4 | 7.2 | 11.9 | 19.7 | 32.1 |
| 0.20 | 1.20 | 1.48 | 1.94 | 2.8 | 4.2 | 6.7 | 10.9 | 17.5 | 27.6 |
| 0.25 | 1.20 | 1.48 | 1.92 | 2.7 | 4.1 | 6.4 | 10.0 | 15.7 | 24.2 |
| 0.30 | 1.20 | 1.48 | 1.91 | 2.7 | 4.0 | 6.0 | 9.3 | 14.2 | 21.6 |
| 0.35 | 1.20 | 1.47 | 1.89 | 2.6 | 3.8 | 5.7 | 8.7 | 13.1 | 19.5 |
| 0.40 | 1.20 | 1.47 | 1.88 | 2.6 | 3.7 | 5.5 | 8.2 | 12.1 | 17.8 |
| 0.45 | 1.20 | 1.47 | 1.87 | 2.5 | 3.6 | 5.3 | 7.7 | 11.2 | 16.3 |
| 0.50 | 1.20 | 1.46 | 1.85 | 2.5 | 3.5 | 5.0 | 7.3 | 10.5 | 15.1 |
| 0.55 | 1.20 | 1.46 | 1.84 | 2.5 | 3.4 | 4.9 | 6.9 | 9.9 | 14.1 |
| 0.60 | 1.19 | 1.46 | 1.83 | 2.4 | 3.3 | 4.7 | 6.6 | 9.3 | 13.2 |
| 0.65 | 1.19 | 1.45 | 1.82 | 2.4 | 3.3 | 4.5 | 6.3 | 8.9 | 12.5 |
| 0.70 | 1.19 | 1.45 | 1.81 | 2.4 | 3.2 | 4.4 | 6.1 | 8.4 | 11.8 |
| 0.75 | 1.19 | 1.45 | 1.80 | 2.3 | 3.1 | 4.2 | 5.8 | 8.0 | 11.2 |
| 0.80 | 1.19 | 1.44 | 1.79 | 2.3 | 3.1 | 4.1 | 5.6 | 7.7 | 10.6 |
| 0.85 | 1.19 | 1.44 | 1.77 | 2.3 | 3.0 | 4.0 | 5.4 | 7.4 | 10.2 |
| 0.90 | 1.19 | 1.44 | 1.76 | 2.3 | 2.9 | 3.9 | 5.2 | 7.1 | 9.7 |
| 0.95 | 1.19 | 1.44 | 1.75 | 2.2 | 2.9 | 3.8 | 5.1 | 6.8 | 9.3 |
| 1.00 | 1.19 | 1.43 | 1.75 | 2.2 | 2.8 | 3.7 | 4.9 | 6.6 | 9.0 |

Go Back

Carbonation tests were carried out in a laboratory by which the equivalent concrete thickness $R_b$ was determined for each coating system. The rate of deterioration of the coatings was evaluated by test results and experience.

According to the results of calculations the organic coatings proved to be most effective with respect to carbonation resistance. The coefficient of service life by some organic coatings was as high as 10.

## 3 SUMMARY AND CONCLUSION

The effects of coatings on the service life of concrete facades were studied by computer simulation. The parameters of moisture transfer properties of concrete were modified by reduction factors which were determined by simple laboratory tests with coated and uncoated concrete specimens. The rate of degradation was also evaluated based on tests and experience.

From the results of computer simulation, recommendations for the service life design were given. The coating factors of service life were determined both with respect to frost damage and carbonation. The coating factor expresses the relative extension of service life of a coated facade as compared to the service life of an uncoated facade. The designer can choose the recoating frequency but it affects the coating factor of service life. The recommendations for service life design are general and are based only on functional properties of coatings.

Organic coatings that separate the active capillary pores of concrete from the exposed surface may protect the structure effectively against frost damage. However, they still must be permeable to water vapour. According to calculations the most effective protection against frost damage was provided by water repellent impregnation agents. They set an effective barrier to the rain water but only slightly retard the evaporation of water from the structure.

All coatings deteriorate with time. So frost damage will appear inevitably in concrete if the concrete is not frost resistant and not recoated in time. The recoating period must be shorter than the service life of the coating. The safety margin, which is defined as the relation of the evaluated service life and the recoating period, is the factor by which the effect on the service life with respect to frost damage of the structure is evaluated. By impregnation agents and some organic coatings the coating factor may be as high as 6 - 8.

Almost all coatings lengthen the service life of a concrete facade with respect to carbonation. However, much variation is found in the effectiveness of protection. Cement based materials with a low polymer content and impregnation agents were least effective. By such coatings the service life factor with respect to carbonation was between 1 and 2 while by some organic coatings it was more than 10.

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# Aging Effect Of External Building Materials

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**Summary: It is important to maximize the design life of buildings in response to the increasing awareness of environmental issues. Consequently, there is now a demand for the aesthetic qualities of buildings to be maintained for a long period. The purpose of this study is to examine the relationship between the color properties of brickwork and the subjective evaluation of age and other visual qualities. The influence of color coordination on aging and appeal is clarified through the results of sensory tests and systematized for the long-term management of building facades.**

**Keywords: external building materials, aging, bricks finishing, color properties, sensory test**

## 1 INTRODUCTION

Knowledge of environmental issues has increased the demand for techniques for managing and preserving the earth's resources, and this is particularly the case in the construction industry, which has witnessed a demand for an increase in the design-life of structures. The Architectural Institute of Japan established a charter for the improvement of the design life of buildings (Architectural Institute of Japan 2000). However, it is only possible to improve the durability of buildings through the implementation of effective maintenance systems. Such techniques should also consider the preservation of the aesthetic qualities of the structure. Many advanced city formations with a high visual impact have been constructed and the importance of the color of the buildings forming a townscape has been established. Especially, variation in color is an important factor influencing the aesthetic appearance of a development. It is therefore of vital importance for the aesthetic qualities to be maintained for as long as possible if the life-span of the development is to be maximized.

Inagaki (1996), Nakamura *et al.* (1997) and Kita *et al.* (1997) showed that color coordination and planning influences the evaluation of the appearance of buildings. However, these studies only examined the state of new or recently repaired buildings and there was little evaluation of cases in which the surface condition of the building has changed with time. Okajima *et al.* examined the time dependent change in Jingu shrines (Okajima *et al.* 1995) and evaluated the surface finish of concrete buildings over time (Okajima *et al.* 1998). However, an evaluation of the color of various external building materials has not been made. The present authors have established patterns in the aging of a wide variety of designs and periods of buildings and some external building materials were identified as being appealing in all eras (Kitsutaka & Kamimura 1990). The aesthetic qualities of buildings will be improved through effective consideration of the color properties of these materials.

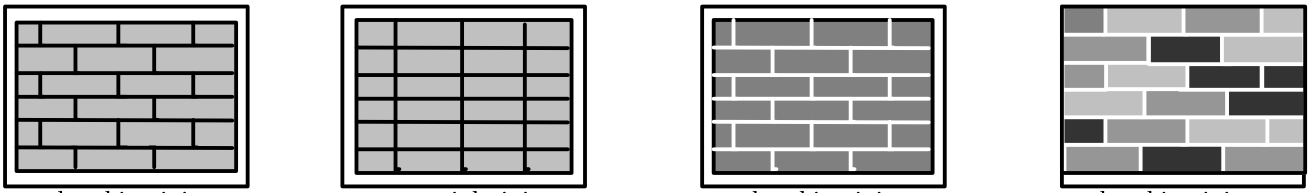
The purpose of this paper is to examine the effects of aging, which is defined as the time-dependent improvement in scene-quality of landscape materials. The effect of the color properties of bricks on the evaluation of aging is demonstrated. Sensory tests on the classification of age and several visual qualities were performed by varying the color properties and bond in images of brickwork samples.

## 2 INFLUENCE OF COLOR ON THE APPEARANCE OF AGE

It was proposed by the present authors that it is a simple process to achieve the effect of aging on the surface of bricks (Kitsutaka *et al.* 2000). A range of samples were used in this project to study the influence of color coordination and joint composition on the appearance of age

### 2.1 Sensory tests

Images of brickwork samples with both breaking and straight joints were prepared and the hue varied in five stages, the Munsell value (hereafter denoted value) and chroma in ten stages and the color of the mortar set as either black or white. Figure 1 shows examples of the images. The following procedure was followed for the preparation of the images. Photographs of brickwork samples were scanned, and the L*, a*, and b* values varied using the Adobe PhotoShop software. The images were output by a color LASER printer. The images 120 mm × 170 mm, the individual bricks were 28 mm × 80 mm, and the thickness of the mortar was 10 mm. The color of each image was visually inspected to check reproducibility. The sensory test was performed on 70 images by subjects who were all architectural students (19 men & 18 women with normal eyesight and 20 years old). The laboratory was arranged such that the images were illuminated with 500 lux fluorescent light (5000 K). Subjects were asked to estimate the age, mood and appeal of each image, and the results quantified using the Scaling of Successive Categories method. Table 1 shows the color information and results for each image. A significant difference was observed in each set of results. The maximum standard deviation was approximately 1.



| breaking joint<br>black mortar<br>uniform color | straight joint<br>black mortar<br>uniform color | breaking joint<br>white mortar<br>uniform color | breaking joint<br>white mortar<br>variegated |
|---|---|---|---|

**Figure 1. Examples of images used in sensory tests**

**Table 1. Color information and measurement results**

| No. | Joint kind | Joint mortar color | JIS Notation | Uniformity of Color | Munsell HVC | Age measure value | Age standard deviation | Mood measure value | Mood standard deviation | Appeal measure value | Appeal standard deviation |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | breaking joint | black | Purplish Red ( pR ) | uniformity | 7.5RP 9/2 | 2.42 | 1.06 | -0.97 | 1.05 | -0.87 | 0.91 |
| 2 | | | | | 7.5RP 6/2 | 1.11 | 1.06 | 0.77 | 1.29 | -0.24 | 1.01 |
| 3 | | | | | 7.5RP 3/2 | -2.79 | 1.13 | 3.66 | 0.73 | 2.12 | 0.84 |
| 4 | | | | | 7.5RP 8/6 | 3.72 | 0.74 | -2.36 | 0.80 | -3.51 | 1.02 |
| 5 | | | | | 7.5RP 6/6 | -0.95 | 0.80 | 0.52 | 1.06 | 0.24 | 0.97 |
| 6 | | | | | 7.5RP 3/6 | -1.67 | 1.13 | 1.26 | 1.26 | 0.19 | 1.07 |
| 7 | | | | | 7.5RP 7/10 | 3.06 | 0.91 | -2.70 | 0.82 | -2.67 | 1.05 |
| 8 | | | | | 7.5RP 6/10 | 2.84 | 0.99 | -3.25 | 0.69 | -3.92 | 0.98 |
| 9 | | | | | 7.5RP3/10 | 2.34 | 0.91 | -3.71 | 0.57 | -4.40 | 0.91 |
| 10 | | | | | 7.5RP 5/14 | 3.25 | 1.00 | -3.89 | 0.46 | -4.66 | 0.94 |
| 11 | | | Red ( R ) | | 5R 9/2 | 2.36 | 0.91 | -1.01 | 0.91 | -0.92 | 1.01 |
| 12 | | | | | 5R 6/2 | 0.86 | 0.96 | -0.04 | 1.06 | -0.93 | 1.09 |
| 13 | | | | | 5R 3/2 | -2.27 | 1.18 | 3.30 | 0.88 | 2.10 | 1.06 |
| 14 | | | | | 5R 7/6 | 2.78 | 1.04 | -1.83 | 0.69 | -2.68 | 1.06 |
| 15 | | | | | 5R 6/6 | 0.35 | 0.92 | 0.67 | 0.81 | 0.95 | 0.95 |
| 16 | | | | | 5R 2/6 | -1.84 | 1.00 | 2.60 | 0.88 | 1.82 | 0.86 |
| 17 | | | | | 5R 7/10 | 2.20 | 0.84 | -1.89 | 0.95 | -2.47 | 1.08 |
| 18 | | | | | 5R 5/10 | 2.26 | 0.85 | -1.69 | 0.84 | -1.48 | 0.89 |
| 19 | | | | | 5R 3/10 | 1.95 | 0.91 | -2.15 | 0.97 | -2.18 | 0.97 |
| 20 | | | | | 5R 5/14 | 2.95 | 1.04 | -3.89 | 0.46 | -4.72 | 0.91 |
| 21 | | | Yellowish Red ( yR ) | | 7.5R 9/3 | 2.03 | 0.77 | -0.74 | 1.01 | 0.13 | 0.99 |
| 22 | | | | | 7.5R 6/3 | 0.49 | 1.07 | 1.70 | 0.88 | 1.31 | 1.02 |
| 23 | | | | | 7.5R 3/3 | -3.07 | 1.09 | 3.80 | 0.55 | 3.14 | 0.81 |
| 24 | | | | | 7.5R 7/6 | 2.48 | 0.80 | -1.12 | 0.78 | -1.30 | 0.83 |
| 25 | | | | | 7.5R 6/6 | 0.39 | 0.78 | 0.84 | 0.75 | 1.99 | 0.81 |
| 26 | | | | | 7.5R 3/6 | -1.50 | 0.68 | 2.67 | 0.77 | 2.29 | 0.80 |
| 27 | | | | | 7.5R 8/10 | 1.45 | 0.63 | -0.34 | 0.74 | -0.10 | 0.66 |
| 28 | | | | | 7.5R 6/10 | 1.23 | 0.98 | -0.65 | 1.00 | -0.44 | 0.99 |
| 29 | | | | | 7.5R3/10 | 0.42 | 0.98 | -0.68 | 0.72 | -0.98 | 0.71 |
| 30 | | | | | 7.5R 6/14 | 1.96 | 1.00 | -1.65 | 0.85 | -2.03 | 0.85 |
| 31 | | | Orange ( O ) | | 5YR 9/3 | 1.95 | 0.85 | -0.02 | 0.90 | 0.59 | 0.87 |
| 32 | | | | | 5YR 6/3 | 0.37 | 0.91 | 0.96 | 0.81 | 1.26 | 0.80 |
| 33 | | | | | 5YR 3/3 | -3.07 | 0.82 | 2.66 | 1.07 | 2.05 | 0.91 |
| 34 | | | | | 5YR 9/6 | 2.64 | 0.88 | -1.14 | 0.87 | -1.57 | 0.81 |
| 35 | | | | | 5YR 7/6 | 1.16 | 0.68 | 0.14 | 0.92 | 0.58 | 0.96 |
| 36 | | | | | 5YR 3/6 | -1.28 | 0.88 | 1.81 | 0.76 | 2.54 | 0.93 |
| 37 | | | | | 5YR 8/10 | 1.53 | 0.93 | -1.76 | 0.99 | -1.78 | 0.79 |
| 38 | | | | | 5YR 6/10 | 0.95 | 0.91 | -0.50 | 0.69 | -0.35 | 0.79 |
| 39 | | | | | 5YR 4/10 | 0.20 | 0.96 | 0.82 | 0.81 | 1.23 | 0.84 |
| 40 | | | | | 5YR 6/14 | 1.62 | 0.93 | -1.95 | 0.79 | -2.77 | 0.74 |
| 41 | | | Reddish Yellow ( rY ) | | 10YR 9/3 | 2.55 | 0.93 | -2.31 | 0.91 | -2.76 | 0.95 |
| 42 | | | | | 10YR 6/3 | 0.21 | 0.76 | 0.47 | 0.95 | -0.11 | 1.03 |
| 43 | | | | | 10YR 3/3 | -2.62 | 0.95 | 2.80 | 0.87 | 1.28 | 0.97 |
| 44 | | | | | 10YR 9/6 | 2.92 | 1.01 | -2.65 | 0.99 | -3.15 | 0.98 |
| 45 | | | | | 10YR 8/6 | 1.99 | 0.79 | -0.82 | 0.77 | -0.80 | 0.91 |
| 46 | | | | | 10YR 6/6 | -0.29 | 0.88 | -0.03 | 0.89 | -0.99 | 0.81 |
| 47 | | | | | 10YR 8/10 | 3.42 | 0.80 | -3.59 | 0.52 | -4.01 | 0.85 |
| 48 | | | | | 10YR 7/10 | 3.18 | 0.83 | -3.29 | 0.81 | -3.92 | 1.00 |
| 49 | | | | | 10YR 6/10 | 0.86 | 0.83 | -1.14 | 0.82 | -2.15 | 0.76 |
| 50 | | | | | 10YR 8/14 | 3.12 | 0.82 | -3.55 | 0.58 | -4.20 | 0.97 |
| 51 | breaking joint | white | pR | | 7.5RP 3/6 | -1.17 | 0.99 | 1.68 | 0.95 | 0.09 | 0.84 |
| 52 | | | R | | 5R 2/6 | -2.08 | 0.97 | 2.48 | 0.80 | 1.22 | 0.90 |
| 53 | | | yR | | 7.5R 3/6 | -0.74 | 0.76 | 0.12 | 0.97 | 0.28 | 0.91 |
| 54 | | | O | | 5YR 3/6 | -1.96 | 0.85 | 2.20 | 0.94 | 1.85 | 0.89 |
| 55 | | | rY | | 10YR 6/6 | -0.40 | 1.01 | -0.40 | 0.86 | -1.68 | 0.84 |
| 56 | straight joint | black | pR | | 7.5RP 3/6 | -0.63 | 1.09 | 1.83 | 1.19 | 1.37 | 1.04 |
| 57 | | | R | | 5R 2/6 | -2.79 | 0.97 | 2.06 | 1.01 | 1.53 | 1.09 |
| 58 | | | yR | | 7.5R 3/6 | -0.87 | 0.90 | 1.62 | 0.74 | 0.99 | 0.82 |
| 59 | | | O | | 5YR 3/6 | -0.83 | 0.83 | 1.55 | 0.73 | 2.06 | 0.77 |
| 60 | | | rY | | 10YR 6/6 | -1.08 | 0.90 | -0.38 | 0.87 | -1.91 | 0.78 |
| 61 | straight joint | white | pR | | 7.5RP 3/6 | -1.42 | 1.02 | 2.50 | 0.79 | 0.65 | 0.89 |
| 62 | | | R | | 5R 2/6 | -1.67 | 0.90 | 2.25 | 0.93 | 1.61 | 0.98 |
| 63 | | | yR | | 7.5R 3/6 | -1.00 | 0.95 | 0.59 | 0.89 | -0.70 | 0.90 |
| 64 | | | O | | 5YR 3/6 | -1.24 | 0.86 | 1.74 | 0.76 | 1.52 | 0.92 |
| 65 | | | rY | | 10YR 6/6 | -2.17 | 0.82 | 0.78 | 0.98 | -0.80 | 0.87 |
| 66 | breaking joint | white | pR | variegated | H : 7.5RP, 5R 7.5R, 5YR, 10YR V : 2ü 6 | -2.24 | 1.18 | 0.45 | 1.41 | -0.94 | 1.15 |
| 67 | | | R | | | -3.43 | 0.88 | 1.20 | 1.32 | -0.65 | 1.03 |
| 68 | | | yR | | | -1.79 | 1.07 | -1.41 | 1.06 | -2.16 | 1.09 |
| 69 | | | O | | | -0.94 | 1.05 | -1.80 | 1.06 | -0.23 | 1.12 |
| 70 | | | rY | | | -2.06 | 1.10 | -1.66 | 1.11 | -1.81 | 1.03 |
| Average | | | | | | | 0.92 | | 0.88 | | 0.92 |

## 2.2 Sensory test result and consideration

### 2.2.1 Influence of hue, value & chroma

Figure 2 shows the results of the evaluation of the age of brickwork with a breaking joint, black mortar, five hues and ten values & chromas. With regard to the high value area, a peak age estimate was observed for the middle chroma but the overall trend suggests that the subjects thought that the sample was new. Although as for the middle and low value areas, they were increasingly classified as new as the chroma increases, the rate of change for a low chroma was large. The middle value, which had a large variation in hue at middle values and low chromas, was evaluated at such an old age that the hue approaches purple and red. The high value samples were classified as new structures and the middle value samples were classified as being slightly older, as were the low value samples with a high chroma.

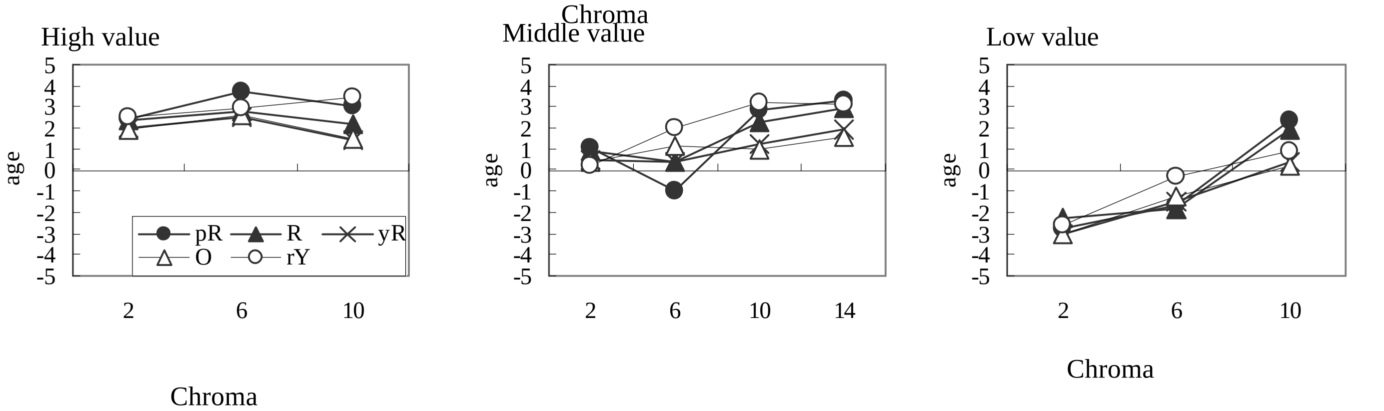



**Figure 2. Evaluation of the age of brickwork** (breaking joint and black mortar)

Figure 3 shows the results of the evaluation of the mood of the brickwork samples analyzed in Fig. 2. The results suggest a decreasing trend with increasing chroma. All high chroma samples were evaluated as restless. The low chroma samples contained a significant variation in hue at low value and high chroma, and were evaluated as restless.

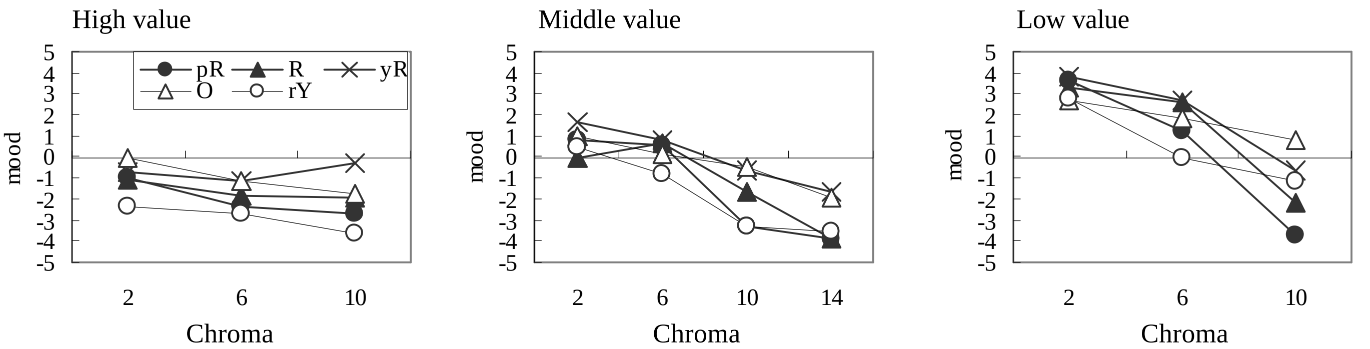



**Figure 3. Evaluation of the mood of brickwork** (breaking joint and black mortar)

Figure 4 shows the results of the evaluation of the appeal of the samples analyzed in Fig.2. With regard to the high value samples, valley-type variations were observed but the general trend was to evaluate the sample as not appealing. With regards to the middle and low value samples, decreasing appeal with increasing chroma was observed with approximately equal rates of change. The low value area, which exhibited a significant variation in hue at high chroma, was generally classified as appealing.

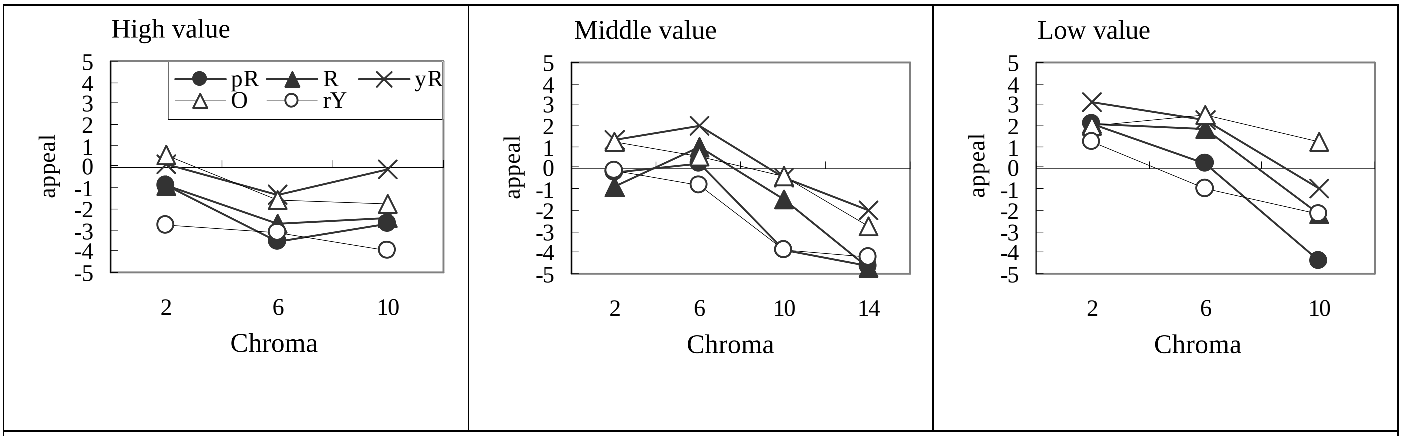



**Figure 4. Evaluation of the appeal of brickwork** (breaking joint and black mortar)

Figure 5 shows the relationship between appeal and the estimate of age for brickwork with a breaking joint and black mortar and Fig.6 shows the relationship between appeal and mood. Both the estimate of age and the mood show a high correlation with appeal and this was particularly the case for samples with a yellowish red (yR) and orange (O) hue. The optimum appeal was achieved for samples with a low value and low chroma of orange (O) hue, and the lowest was for an average value and high chroma of purple red and red. Figure 7 shows the tendency for each hue, and the relationship between value & chroma and age, mood, and appeal. In all hues, samples with a low value and low chroma were classified as old, mood, and appealing. Samples were also classified as appealing when the reddish yellow (rY) hue was removed, as in the middle value and middle chroma, and the low value and middle chroma samples. In addition, the range of samples classified as appealing was broad when an orange (O) hue was added. Samples with a reddish yellow (rY) hue and low value and low chroma were classified as appealing.

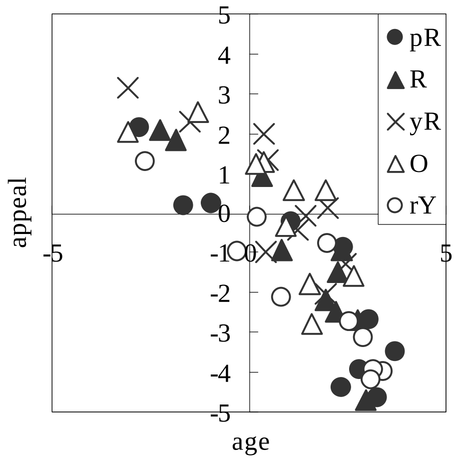



**Figure 5. Relationship between appeal and age**
(breaking joint and black mortar)

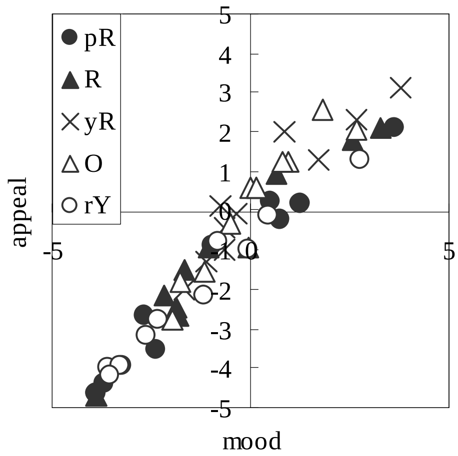
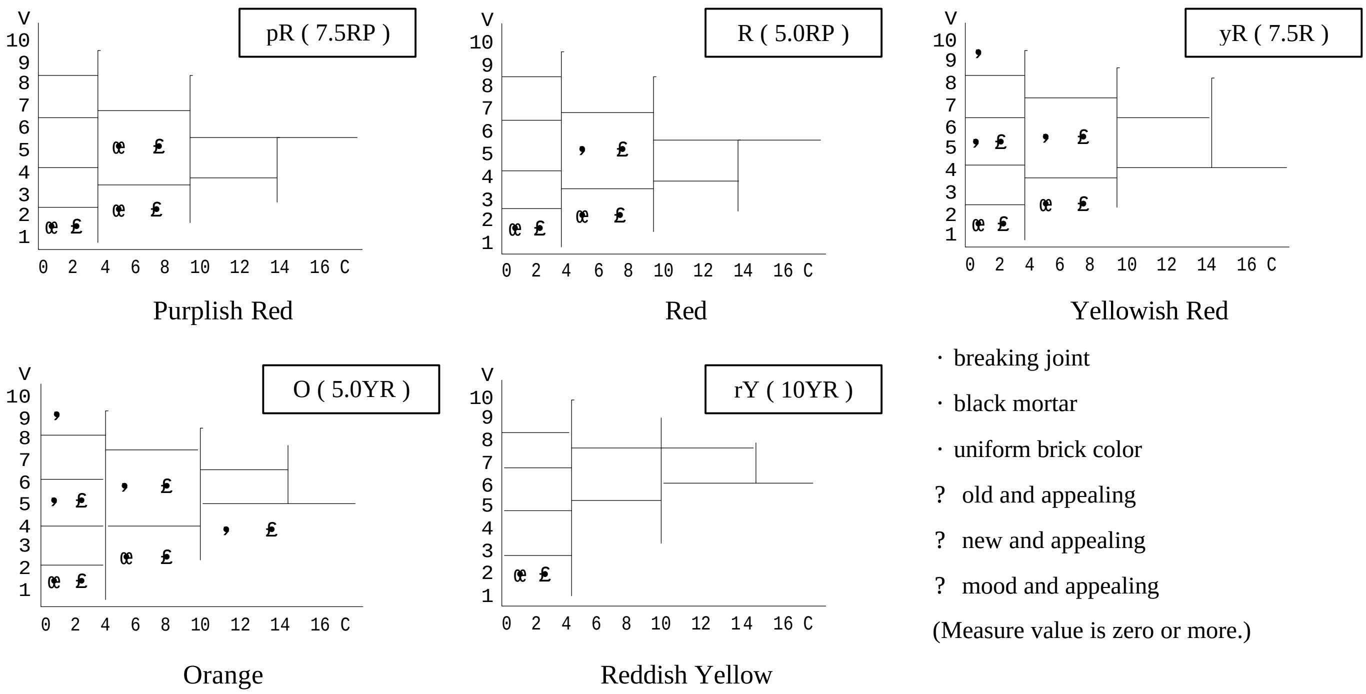



**Figure 6. Relationship between appeal and mood**
(breaking joint and black mortar)




· breaking joint
· black mortar
· uniform brick color
? old and appealing
? new and appealing
? mood and appealing
(Measure value is zero or more.)

**Figure 7. Relationship between age mood, and appeal for each hue, value & chroma**

### 2.2.2 Influence of joint

Figure 8 shows the results of the classification of the appeal of samples with five hues and color tone unification. Figure 9 shows the evaluation of the mood and Fig. 10 shows the evaluation of the age. The variegated samples were classified as not appealing and restless in every hue. Only the samples with a reddish yellow (rY) hue were classified as not appealing in all states. Therefore, bricks with this hue, low value and middle chroma are regarded as not suitable for final finishes. Bricks with a breaking joint were generally given high classifications, and black mortar was favored and classified as mood. Moreover, samples with a reddish yellow (rY) hue and jointed with a white mortar were generally classified as being older than those with a jointed with a black mortar. For other hues however, samples with black mortar were classified as being older than those with white mortar.

? breaking joint, black mortar, uniform color ? breaking joint, white mortar, uniform color

? straight joint, black mortar, uniform color ? straight joint, white mortar, uniform color

× breaking joint, white mortar, variegated ( color tone : dark )

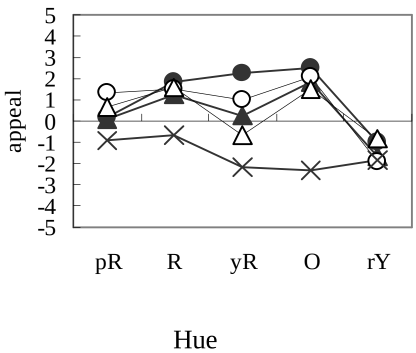



**Figure 8. Evaluation of appeal**

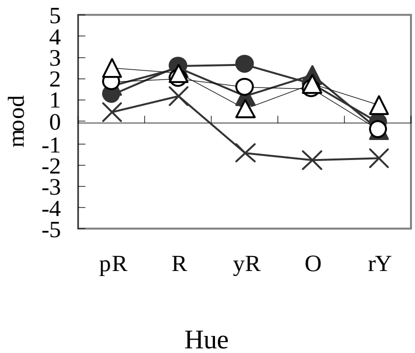



**Figure 9. Evaluation of mood**

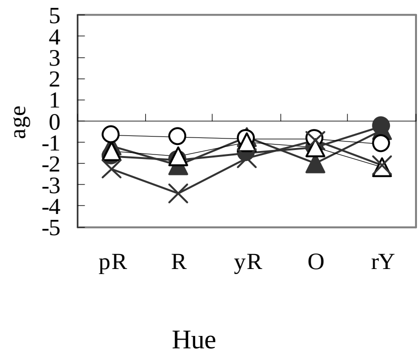



**Figure 10. Evaluation of age**

## 3 USE OF COLOR PROPERTIES IN BRICKWORK DESIGN

The relationship between the color properties of brick finishes and appeal is shown in Fig. 6. It is suggested that regular maintenance of bricks with a high value is needed in order to maintain appeal and mood. Since deterioration, discoloration, and fading of exterior walls cannot be avoided, it is suggested that finishes with a middle value and middle chroma of yellowish red (yR) or orange (O) hue are suitable for most exterior applications. Moreover, since the estimation of age is related to the color of the joint, the appeal and mood are related to the uniformity and the color of the joint. Such considerations must be made when designing exterior features.

## 4 CONCLUSIONS

The conclusions of this study are as follows.

1) Value and chroma were closely related to the subjective evaluation of the age, mood, and appeal of brickwork, and bricks with a low value and low chroma, or low value and middle chroma were identified in a positive manner.

2) The age and mood of brickwork showed a high correlation with appeal.

3) Brickwork with orange and yellowish red hues were classified as appealing.

4) The estimation of the age of brickwork was influenced by the color of the joint, and appeal and mood were influenced by the type of joint used.

5) A breaking joint was consistently classified as appealing.

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# Relifing Constructed Works

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**Summary: Relifing is an integral part of life care management. It provides an opportunity to respond to service loss, however caused, and may be used to address serviceability shortcomings of a given constructed work or its constituent parts. Relifing will also contribute to the decision making in service life planning terms, and encourage one to sustain and extend the 'first life' of a constructed work, or its respective service lives. In particular, by encouraging intervention through appropriate life based management strategies and practices, or upgrading or restructuring as the case may be. Such responses may also be seen as sustainable actions, if only to keep the embodied energy laid up in service.**

**But, relifing constructed works, or their constituent parts, and responding to obsolescence deficiencies, or varying levels of satisfactory and unsatisfactory performance, present a number of practical problems that must be resolved. Firstly, addressing business functionality, space, operability and reliability quality and its serviceability response. Secondly, optimising upon the service life mixes of the materials and components service life profiles in place or proposed. Thirdly, establishing the constructed work's service life or its life cycle assessment relationships, or what is proposed, and their service life years.**

**Relifing justifications may cause conflict with some aspects of sustainable policy or practices and regulatory requirements. So the paper discusses relifing, its measurement issue, and the feedback for service life planning, as well comments upon 'sustainable practice' through LCA. In doing so, suggesting the need to introduce both the construct of the service life year and service life cycle assessment so as to encourage wholelife cost development.**

**Consideration is given to the importance of performance, and in particular the performance level. At the same time the link and contribution of performance is made, concluding with the need to encourage performance condition based monitoring. The paper stresses that the constructed work’s service life cycle assessment requires an appropriate context- both for wholelife thinking and life care management. Here, too rigid an interpretation of environmental, performance and/or statutory requirements may weaken relifing’s contribution to whole-life care practices, or reducing wholelife costs. Especially, where rebuilding may become more practical or necessary in order to meet that fitness for purpose of the client despite conflict with sustainability intentions in recycling terms.**

**As well as relating to those decisions concerning service life cycle assessment, relifing is seen as an essential part of service life planning in all new projects as well as many existing constructed works. In a sustainability sense too, the future for better building should encourage taking better care of the built environment’s constructed works through relifing and encouraging their re-use as the case may be. The paper sets out a flow chart indicating the relationship of relifing within service life planning. Concluding, it is suggested that relifing should be seen as an integral part of ISO 15686 and whole and service life planning, and, contribute to whole life cost and its care management for sustaining new and existing works.**

**Keywords. Performance level service life, relifing, service life cycle assessment, service life years.**

## 1 INTRODUCTION

The pursuit of sustainable practices through life cycle assessments, tend not to address the whole life performance of new constructed assets or facilities, nor their service life management. Nor are those responses to the problems of obsolescence, or how one is to adequately address the problems arising from changing performance standards over time made explicit. Yet, it is with the existing built environment that urgent attention should be directed. Both in order to respond to the recovery of the loss of serviceability in constructed works, and manage the potential supply chains of materials and components when kept on station or directly incorporated in changed constructed works.

Where recycling occurs it is past a decision point in an environmental sense, where the embodied energy commonly found in existing infrastructure and other constructed works, like the neglected but redundant and obsolete building, shown in Figure 1 below, can, or could be reused and kept on station. Simply, life cycle assessment has yet to adjust to, or take on board the place and significance of the service life. Further, there remains an inadequate acknowledgement that all systems are subjected to a transformation process, have a life cycle, a service life and an end date.

**Figure 1. An obsolete Building, with a significant embodied energy stock.**

In one sense too, relifing, the main focus of this paper can be seen as part of an environmental response for taking care of what we have. So it should be an integral part of wholelife management! New uses too may be found for redundant or, obsolete constructed works, especially where regeneration is part of a wider strategy than the immediate site (W/E Consultants Sustainable Building 1999). So the case for '*Increasing environmental efficiency, which can be achieved by prolonging the life cycle of the building and its components'* has already been made (Durmiseviv & Heystplantsoen 2000. Further, the researchers also suggest that focus should be directed to also securing space flexibility). Such possibilities are less likely to occur, however, without prescription, or some form of concession that can in turn be offset against the embodied energy retained and costs incurred.

In fact, service life engineering aspects relating to both sustainability and especially existing constructed works have yet to become integrated within a recognised common service life *or relifing framework*. There is also a danger too; that the existing embodied energy may be wasted outside a limited recycling unless a replacement strategy related to age, condition or performance level can be encouraged and a simple universal measurement base agreed. Further experience of measuring environmental or performance generally, must be tinged with caution. (Lippiat and Boyles 2001) As indeed the nature of data capture, and assessment so that those who have to design can use it, specify or manage life based constructed works.

## 2 RELIFING

The ground rules for relifing and the associated life-care need should be developed within a performance context. And in turn, related to a common whole life cost framework, and the portfolio or regulatory standards in force at the time. Nevertheless, the loss of service of a constructed work or facility may in a wider sense, be recovered through some form of intervention.

In many ways, one may come to see relifing as a response to address service losses, however caused. Inevitably, conflict may exist between the 'nobler intent' of optimising durability attributes of a component or material's service life, as any life design is the sum of its systems, and their range of performances with portfolio or business needs. But what of a wholelife performance where service life and relifing can come together?

Indeed, the construct of whole-life performance (Rowe 1998) supports the case for relifing! In consequence, it is suggested that a lifetime scope is needed especially *if* service planning is to play a part in relifing. Yet, in relifing terms, one must also address the service life attributes of the parts in the context of the asset and its facilities operability, as well as their serviceability attributes. In such a situation, one might then talk of extending the original service life by relifing, an action especially encouraged when pursuing sustainability practices, or seeking to optimise on lifetime value- a point briefly discussed later.

Before proceeding further, it might also be helpful to define, what is meant by the term relifing in this paper. Relifing is taken to mean, '*a strategy or course of action adopted, whether by intervention or change, that extends, or sustains a service life existing, or a course proposed, for the designed life intended of a constructed work, or, its constituent parts. In both cases, only when it is economic in a wholelife cost context to do so.* (Lucchini and Wyatt 2001)

Relifing as such does not have a recognised, nor common quantitative framework. Instead, it may be viewed more in terms of being qualitative with an underpinning of quantitative inputs. Nevertheless, relifing should be seen as a powerful mindset when;

- Evaluating, existing portfolio strategies, for example, the replacement of less efficient constructed works or sub-standard facilities
- Establishing the existing performance state or condition of a constructed work
- Carrying, out the evaluation of an existing constructed work's facility surfaces, or space
- Responding, to the performance based condition of a system or systems, of a constructed work
- Responding, to the performance based quality, or condition of the constructed work's component(s)
- Considering, environmental life cycle assessments of a given service life or lives
- Appraising, the environmental load of an existing constructed work and/or its core structure and elements in order to establish an appropriate amelioration strategy
- Reviewing, the recovery of given systems, plant, equipment, components and some materials for reuse on, or of site in another constructed work
- Using, service life planning to establish life care management responses in new works, or projects relating to existing works.

At the same time, a suitable framework is required, together with measurements that can take into account environmental, performance and functional support needs of the organisation's requirements for the most practical economic outcome sought (Davis G & Szigeti F 1999).

Relifing is applicable to both existing and new developments and especially when dealing with fit outs and the renewal of elements, systems and components, as will be discussed later, under service life cycle assessments. It also has a considerable number of permutations as illustrated in Table1, which may assume significance, both a wholelife cost, and the performance-based approach to the management of a constructed work.

**Table 1. The Constructed Work-Facility Asset Relifing Options**

| | Life care responses | | | | Relifing | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Option+ - response > | Clean | Service | Repair | Replace | Renew | Upgrade | Reconstruct | Restore | Conserve/ Preserve |
| *Space- | | | | | | | | | |
| Element System | | | | | | | | | |
| Component | | | | | | | | | |
| Material | | | | | | | | | |
| Surface | | | | | | | | | |

*It should be noted here (though not discussed) that client business or organisational changes or developments in technology or specification standard requirements, could cause a layout- (space) to become obsolete, or not fit for that purpose. Indeed fitness is acknowledged in Figure 2 and seen as a core value to be robustly developed in relifing and value enhancement.

+Such permutations, or options form an integral part of both Figures 2 and 3 that appear later and are also linked with Table 2 too.

There are also in practice, specific stages in life care terms, where one may intervene and clean or repair and in doing so avoiding the need to either relife a component, element or their material(s). At the same time too, Table 1's reference to, clean, repair and replace may equally be seen at times as much a relifing response, as it is for life care. One must also accept that there are natural agents of decay and neglect, and degradation cycles with distinct steps ranging from initiation, defect and failure to catastrophic failure. If one likes, the steps represent either a service loss or loss of serviceability, in the area under attack and by inference a threat to performance and reliability expectations.

Finally, relifing is seen here as an adjunct of service life, life care and it is believed and discussed later an integral part of service life planning. At the same time, in order to respond to ISO 15686 with its service life and performance focus, the terminology currently used in post commissioning of new work may not necessarily accord with the relifing option model being proposed here.

## 3 ECONOMICS. WHOLELIFE. AND. THROUGH LIFE COSTS

Relifing is more appropriately handled where the whole constructed work can be considered in terms of their economic lifetime performance. In fact, although not the focus of this paper, one could have developed the case for relifing by focussing upon recouping and managing a whole lifetime value, but in an economic sense. (Templemans Plat 2001)

Securing a whole life approach (and by inference, a sustainable environment) two specific issues however, need to be addressed. Firstly, the portfolio or building stock stake and the key holders and their funders outlook or strategy to replacement and life time practice through value that can be economically demonstrated. And secondly, the need to move to a service life *relifing inclusion* with its appropriate life cycle assessment, together in a whole life based performance context (Ang et al 2001) with the *associated through and wholelife costs.*

This focus on economic value may be encouraged, by the need to manage the retention of embodied energy within carbon based credit based value and landfill taxes. So in relifing sense and service life planning too, one considers:

- Integrating life cycle assessment and the service life as *service life cycle assessment*
- A range of relifing options as suggested in Table 1
- Life care requirements for the facility and asset against a given class or performance level
- Goal based reduction of those negative environmental impacts of existing constructed works within the portfolio
- Life time buys
- Disassembly and reassembly relifing system and component development

- Regulatory acceptance of a range of time based and class performance levels of compliance
- Performance based condition assessment and performance based monitoring
- *Through and wholelife cost decision making*
- A lifetime with a definable measurement base or standard that permits a comparative performance analysis to be made, for example, with regulatory but time based multiple levels of-performance (Meacham 2001) existing or proposed with those currently in force or planned for in the future

Meanwhile, the ‘wholelife costs’ is seen as a recognised outcome, in order to facilitate relifing decision making especially at a project or strategic level. This wholelife, cost here is taken to mean:

'Those total costs anticipated, or incurred, of the material and component supply chains used, or required to construct, operate, or maintain a facility, including the operational wholelife time performance and functional levels and associated relifing of the whole constructed work, throughout its’ in service life. And where Service Life Cycle Assessment is considered, the disposal and net recovery through carbon based crediting'.

## 4 SERVICE LIFE CYCLE ASSESSMENT

Service life cycle assessment (SLCA) encourages the LCA aspects of the constructed work’s environmental issues to become an integral part of service life and wholelife and relifing considerations. (Pullen 1998) Yet, establishing the service life cycle assessment relationships, their service life measures, together with relifing justifications, may cause conflict between the estimated costs and the actual cost response. Similarly, the relationship between a constructed work’s constituent parts’ and their respective condition based rating, and, service life *and the service life years to be managed or sacrificed as the case may be different to expected.*

It may help to try to review those options selected from Table 1 and then determine their respective SLCA in a schedule form, as intimated in Table 2. But there is also a risk that in such situations, that such a practice may not prove cost effective, or satisfactory but instead, impractical or unacceptable- especially in terms of current, or likely future regulatory requirement. Nevertheless, where substances become known as being dangerous to health, or are found, they will have to be removed, and landfill charges but for construction waste disposal continue to rise, mending may be better than ending! Relifing then has a place an integral part in waste management and deconstruction.

Even so, life cycle assessments should not discourage a review of those possible benefits by having a new facility, including a more attractive operability requirements, and lower environmental, through-life costs (Gurung and Mahendran 2000).

In practice, a case has to be made to retain a constructed work’s existing embodied energy as well as to address declining performance standards, dereliction, neglect or disposal obsolete construction, systems, plant and equipment in the most practical way. By doing so, ensuring that the net environmental deficit is less than that environmental impact arising from having a new work, or from using a higher percentage of new materials.

Ultimately, two important responses or outcomes are required to complete Table 2’s schedule. Firstly, the ability to assess the actual status for any *performance based condition assessment* (PBCA) for the constructed works constituent parts in a scientific or engineering sense. And secondly, the link with performance levels and the constructed work business and end user fitness for purpose needs, yet not fall foul of environmental regulatory measures.

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**Table 2 Service life cycle assessment**

| PBCA> | Space- | Plant/ Equipment | Systems | Surfaces | Substrate | Components | Elements |
|---|---|---|---|---|---|---|---|
| **Reliability status** | | | | | | | |
| **Serviceability Q** | | | | | | | |
| **Performance level** | | | | | | | |
| **Service life years remaining** | | **MTTR** | **MTTF+** | | | | |
| **Costs of non intervention** | | | | | | | |
| **Disposal costs** | | | | | | | |
| **Option's Costs of intervention** | | | | | | | |
| **Recovery costs** | | | | | | | |
| **Net LCA/SLCA EIA CO2 etc** | | | | | | | |
| **Service life years gained** | | | | | | | |
| **PCBM Costs** | | | | | | | |
| **Thro life costs** | | | | | | | |
| **Whole-life costs** | | | | | | | |

+ Where relevant Here is a considerable body of work in the engineering domain where relifing experiences exist from aircraft engines to ship hulls which can be brought into construction

Relifing research remains to be carried out in a discipline way. In particular, to optimise upon the constructed work's various service lives of its constituent parts and their term or time out cycle. Here the issue of durability, new technologies and methods of reconfiguring existing works may become an integral part of any deconstruction activity and ultimately of design life engineering.

## 5 PERFORMANCE BASED CONDITION ASSESSMENT

The relifing measurement base does, as has been mentioned earlier, raises a number of problems yet to be resolved. Even accepting a range of approaches as necessary, viz a viz survivorship analysis, the Factorial approach (Strand & Hovde 1999. Hovde 2000) and multi-hierarchical durability assessment (Mu and Xianming 1999) there remains a case to consider developing the generic relifing engineering approach as a quick but simple recording activity.

To address this problem, it is suggested that decision trees are used both for the whole work and for its respective elements, systems and their constituent parts, including jointing and finishes. In that sense then, each element and system would have a service life profile, as well as performance settings, which in turn would have a number of service life and service life ratios to balance or manage. (Wyatt and Lucchini 1999) Here, immediately through and wholelife cost can merge with relifing and service life and complement service life cycle assessments and wholelife management.

So, the challenge is to perhaps, develop an in service life based performance approach, that provides a systematic procedure to establish the environmental constructed work's status that is understood. This status, is in effect requiring an evaluation that may range from the whole constructed whole to a discipline breakdown of its systems, elements and given components or materials. The evaluation measures too, are likely to be a value judgement of a qualitative nature, especially in a building sense**.** A similar criticism too, may be levelled with performance based condition assessments (PBCA) or on going or life- time monitoring (PBCM).

Relifing requires an established analytical based approach. The problem, however, is establishing the service life status of the constituent parts in context of the whole in any in relifing cycle. Firstly, how does one establish where in the life cycle point of the curve of any service loss, or if one likes the service life elapsed time of any constituent part since the constructed work was built? Especially, where the systems and components have a different service lives to that of the longest life part of the constructed work, or given components in elements like a roof, heating or cladding system. Secondly, establishing the loss of reliability, for example, of a water heating, or air conditioning system without MTTR or MTTF, or the threat faced to plant and equipment, either due to no life time buy support or spares availability. Worse still, obsolescence may be a bigger threat or regulatory changes. Thirdly, there is the problem of establishing the in-service whole life cost system boundary. (Despite the differing ways relifing and servicing costs are allocated or written down and where little cost consideration is given to relifing or recovery assessment).

Yet, despite significant progress taking place in service life engineered designs, (Siemes 1999) the service life experience of the constructed work's system, component and material feedback format is limited and needs to be kept in mind at this time. If only because the future needs feedback to improve service life planning and life designs as well as life engineering developments for new work to improve predictive approaches!

Carrying out a PBCA in existing constructed works, or their specific constituent parts may present practical problems to relifing goals that in themselves, may neither be wholly affordable, nor always meet with current perceptions of sustainability. One also needs simple-approaches, e.g. for identification of visual referenced based criteria (as has been demonstrated in the damage atlas work and simple recording formats that facilitate computer use) and insitu-tests for evaluating performance status, or service loss.

In relifing, one must also address both the pathology and forensic science and engineering fields, as well as use established techniques and responses in failure developments (Kelly1998). Equally, a quantitative approach can also be brought to bear with components, systems and materials and performance based measurement levels. Here a significant number of researchers from, Shen, Johnson, Huovila, Marshall, to Genge, amongst others, together with, the condition base work of the Norwegians, (NS 3424 1995) have contributed to PCBA. Indeed, this work may now further contribute to the development of service life years and service loss assessments in ISO 15686-7.

## 6 PERFORMANCE BASED CONDITION MONTORING (PCBM)

Again, drawing upon the work of other researchers from Abraham and Wirahadikusumah, Hodges and Siemes and Edvardsen, to Lounis et al, from the BELCAM project team, establishing the predictive rate of service loss, one could secure some precision in PCBM. Further, such work leads to quantification of engineered elements, materials, systems and components in differing, but stated environments and hopefully too, both in their whole works *and their relifing.*

Inevitably, performance requirements will be met with varying degrees of success, and measures of satisfaction, for example, in terms of serviceability, reliability and operating costs. To contribute to the perceived PCBM problem then, it is proposed that the construct of a service life year is introduced and used. Especially where the environmental disposal costs are to be considered, or the accumulated carbon SLCA credits are worth while managing as the carbon index approach now being proposed to tackle global warming is introduced. Service life year is defined as: '*A completed year in use of a component, material or element exposed to change agents or agent in a constructed work, element or system*'.

The basis of client expectations and their responses to a loss of performance, serviceability and budgetary concerns needs to be known. In consequence, performance auditing (ISO/DIS 15686-3 2001) should where practical (Ang and Wyatt 2001) be carried out, both through PCBA and PCBM and relifing responses as intimated in Figure 2 below. Not only to respond to growing environmental and wholelife requirements, but also to seek a pathway through a rigid regulatory system that itself has an insufficient sensitivity to the life cycle aspects of the service and operability lives of an organisation, or the constituent parts of a constructed work. There is also a case for moving to a performance timed based requirement, as demonstrated by the New Zealand's 50year time frame.

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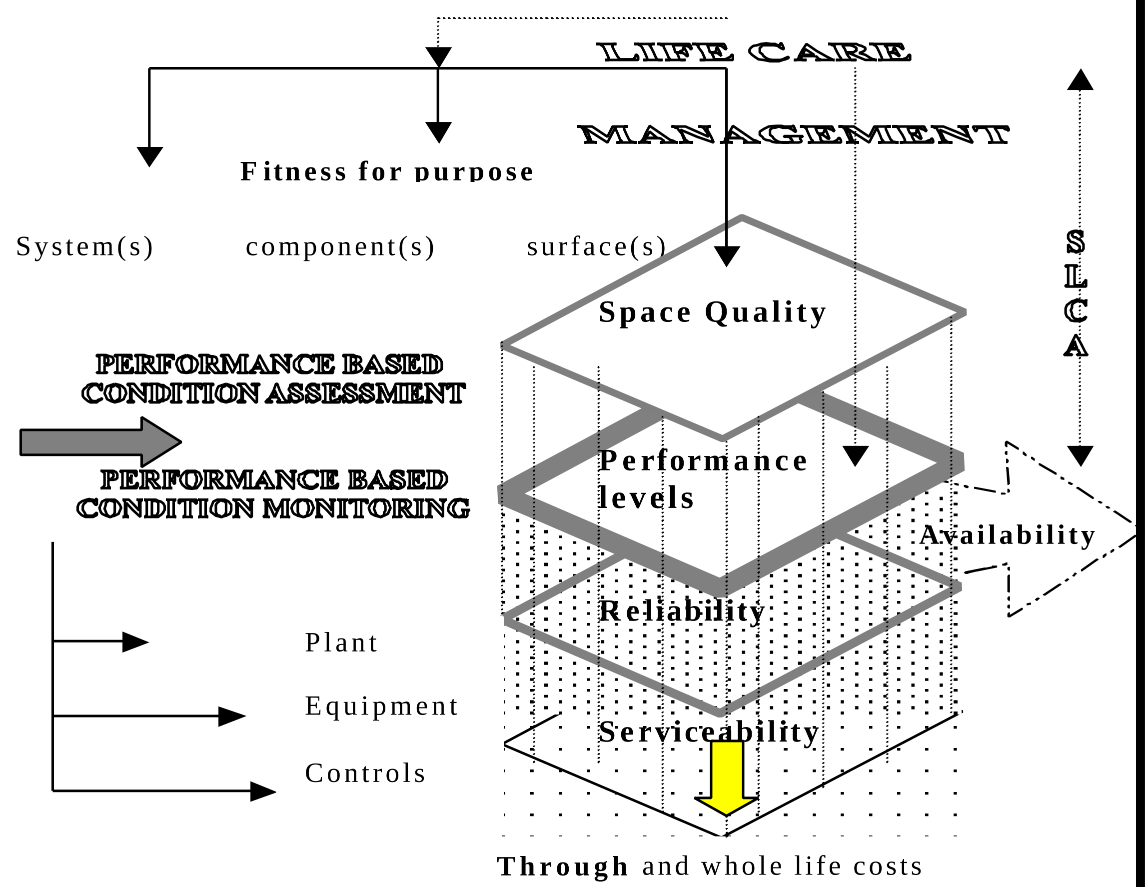


**Figure 2 Performance Based Life Care Management**

To secure both whole life approaches and by inference, a sustainable environment, two specific issues also remain to be addressed. Firstly, the portfolio or building stock stakeholders or key holders and their funders need to move to a service life relifing strategy, and secondly, the life cycle assessment period should be related to *a life-based performance level within a regulatory or advisory standard.* To do so, however, may require a systematic approach for constructed work's performance levels, based upon outcomes to be adopted (Davis & Szigeti 1999: Ang et al 2001)

Nevertheless, the build quality-life remains significantly dependent upon both its external and internal environments, and that life care practised. Yet in managing particularly existing constructed works, one may have levels of acceptability that may not accord with the standard of the day, i.e. though imperfect they are still in use! There may be differing levels of asset or facility practices to acceptance, or tolerance irrespective of any legal consequence, or regulatory requirements, especially, where a constructed work's service life local failings, may spread and affect other areas. So to rigid an interpretation of environmental, performance requirements may weaken relifing's contribution to whole-life care practices. Especially, given the long timescales in replacement stock cycles like housing, hospitals and schools, where significant reconstruction or if rebuilding becomes necessary, or given statutory performance levels compromised. Likewise, where the portfolio or constructed work strategy is to include phasing out given systems, the constructed work's replacement cycle may be slowed down.

Whether addressing existing or new works, however, a number of specific in-service life operational responses or options can be identified that are common to both life care and relifing as intimated in Table 1 earlier. Such options are seen as an integral part of any wholelife decision framework. But it would also assist service life *planning if both service life profiling and decision tress were developed in new and relifing existing work*. Then SCLA and TLC and WLC (Figure 2) will contribute to life care management and optimisation based upon, the service life linked with performance in order to give value. Such performance based outcomes, assist the management of the service life years and help quantify the net changes or sacrifices of service life years as the case may be. Such performances may best be seen in terms of levels of their unacceptability, or as part of a total performance evaluation. (Ang & Wyatt 2001)

It is also a tall order, to move from the separation of design from the post occupational and life care to an acceptance of a whole life management approach, centred upon performance based assessment and monitoring and especially, to include relifing and recovery management. Performance over time and performance levels have already been discussed, but it ultimately it is the need to link both existing and new designs together that must be the goal of service life development. Nevertheless, through developing and adopting a performance audit approach (ISO 15686-3 2001) it is believed that relifing would become to be seen as an important input into ISO 15686.

## 7 SERVICE LIFE PLANNING

Service life planning is concerned with both performance and functional requirements, as well as securing the reliability of the elements, or systems specified or used. Service life planning must consider durability and reliability as well as serviceability through risk and risk management. All relevant aspects need in fact to be an integral part of the designed life's evaluation. It is here, necessary to capture 'the life performance' that will impact upon the relifing or service life design life proposed. So, service life planning should also address the short, the medium *and* the long-term requirements including the range of options for relifing and also focus upon the model representations in Figure 2 and 3.

Nevertheless, to have effective service life planning, one should address the elements and systems constituent parts fully in terms of their probabilistic service life and failure modes. At the same time, their respective service life planning, and criteria should fall fully within the domain of ISO 15686 and its relevant part(s). Here an integrated ISO 15686 Part should also address performance based conditioned base monitoring -including a measurement range from the service life year as well as procedural guidance at two distinct levels. Firstly, the asset part of the constructed work and then, secondly its facility. Both to secure whole and through life costs of ownership based upon quantifiable performance levels that accord with a sustainable recognition and the end user needs and the statutory requirements of the time. If one likes then the future would then be a serviced life engineered and performance managed constructed work. One to be relifed as necessary to give a minimal environmental negative impact and at least whole life cost management. Figure 3 attempts to show how relifing may become an integral part of service life planning.

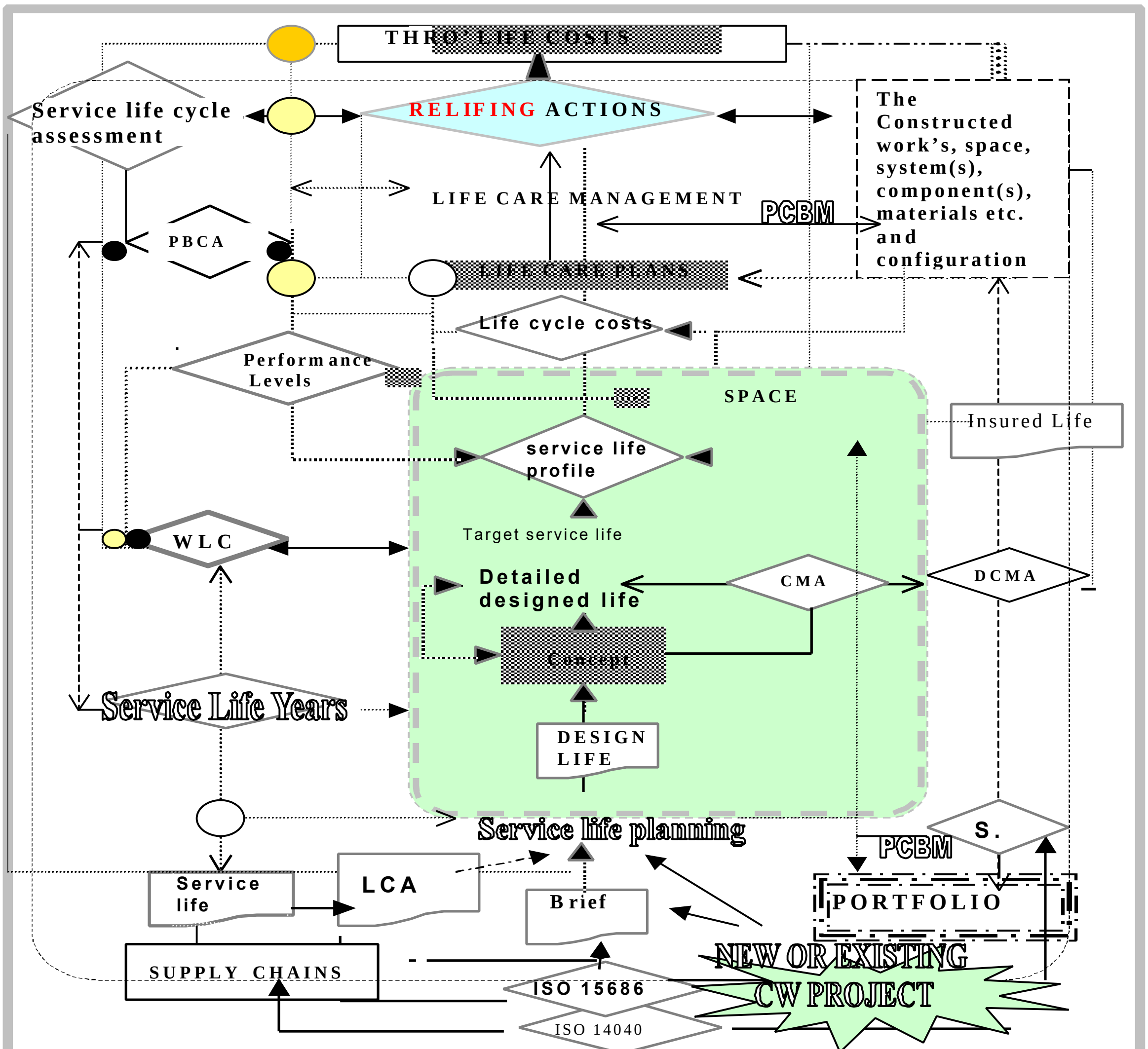


**Figure 3 Whole-Life Development Reflecting ISO 15686 and ISO 14000 Series**

Footnotes

- By driving the wholelife construct through a focus of the service life and its service life cycle assessment in service life planning and life care terms, both through or wholelife costs should be established. In turn when built relying upon both PCBA and PCBM to contribute to feedback and a qualified status for portfolio through to design, value and life care decision-making.
- Equally one must give consideration to extending the performance approach to consider it as a life-cycle performance. In turn this suggest the need consider more fully environmental impacts, in particular on given service life profiles of the different parts of a constructed work. Not only in terms of the service life, but also where relevant assembly (CMA) and disassembly configurations (DCMA) and their embodied energy optimization for the relifing strategies agreed.

As well as relating to decisions concerning sustainable life cycle assessment, relifing should be seen as an essential component part of both service life planning and an integral part of the through and whole life cost model. In consequence, it is to be hoped that the emerging international standard ISO 15686 should be seen as an approach to reducing building ownership and improve wholelife management

## 8 CONCLUSIONS

Relifing will both sustain and extend the 'first life' of a constructed work and its respective service lives through intervention by upgrading or by restructuring management strategies and practices as the case may be. Relifing also provides an opportunity to respond to service loss, however caused, and addresses serviceability shortcomings of any given constructed work or its constituent parts. But it is the need now to try and provide guidance as to how to approach relifing that is of concern. Especially as we move to a probabilistic and yet incomplete whole life quantitative framework.

Establishing through and whole life costs, where optimisation of service life is sort, should take place within a common service life and environmental measurement system. Here, it is proposed that the service life year and service life cycle assessment are also adopted and developed and become part of such as system in time. At the same time, the many important contributions from the growing number of 'dbmc international proceedings' and CIB W080 Commission's help to caution those who seek a single quantitative or only a quantitative approach, and yet encourage the life engineering route to the future.

Indeed, as performance and its life information research continues, life based monitoring engineering approaches need to contribute to wholelife PCBA and PBCM. This in turn will help secure a redundant building’s future- e.g. through a change use and tenancy based on a carbon tax credit exemption, or moving a waste stream's embodied energy to become a supply chain e.g. through disassembly and relifing!

Concluding, in a sustainability sense too, the future for better building must secure better care of the built environment’s constructed works through relifing and its would be hoped too, service life management. But it will remain difficult to persuade the diverse range of stakeholders and their constructed work’s environments, other than through a statutory obligation. Ultimately it is left to those portfolio based clients and developers, together with their project management and design teams, to make minimal demands centred based upon an understanding on the supply chains durability or service life procured and in particular think in terms of wholelife management and wholelife performance.

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# Biodeterioration Of Historic Buildings In Latin America

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**Summary**: **Buildings of cultural heritage are discolored and degraded by the growth and activity of living organisms. Microorganisms form biofilms on the surfaces of stone and painted buildings, with resulting aesthetic and structural damage. The organisms involved are bacteria (including actinomycetes and cyanobacteria), fungi and algae, but protozoa and other small animals are also found. The interactions between these organisms can enhance or retard the overall rate of degradation. In addition, microorganisms within the structure (endoliths) may cause damage. These may grow in cracks and pores within the materials and may bore into rocks such as limestone. True endoliths, present within the rock itself, rather than in voids in the rock, are found in materials such as soapstone and are predominantly bacterial. A review of work on microbial biofilms on buildings of historic interest in various Latin American countries is presented and the microbial activities which lead to degradation of the structures are described.**

**Keywords. Biodeterioration, cultural heritage, microorganisms, weathering**

## 1 INTRODUCTION

Both historic and modern buildings are subject to the deteriorative and degradative action of the environment and living organisms, normally referred to as "weathering". Biological and abiotic processes can occur concurrently, each contributing to the overall deleterious effects, and it can be difficult to determine the contribution of each. However, there is no doubt that biological growths have considerable impact on the soundness of structural materials.


**Fig. 1. Mosses and higher plants growing on limestone building of the Mayan civilization at Sayil, Yucatan, Mexico**

The destructive effects of mosses and higher plants are readily recognized. The root structures penetrate and disrupt the structure of the building (Fig. 1), but plant growth occurs only after a "protosoil" has been produced by the growth and degradative activity of other organisms, less obvious to the naked eye - microorganisms. These form so-called "biofilms" on any humid surface, even if a water layer is not detectable; such biofilms, apart from serving to prepare the surface for plant growth, can themselves have considerable impact on building materials. The problems associated with microorganisms are not familiar to architects and engineers, and are generally disregarded by them; however, microbial activities are potential threats to the maintenance of modern buildings, as well as historic and cultural property. The activities of microorganisms in biofilms are shown in Table 1. Since the organisms are present on the surface of the materials, their activities are localized and concentrated at these points.

**Table 1. The effects of microbial activities on historic buildings**

| Observed effect | Microbial activity | Material(s) | Major microorganisms |
|---|---|---|---|
| Discoloration | Physical presence | All | Algae, cyanobacteria, fungi |
| Retention of water | Physical presence, EPS | All | All |
| Stimulation of growth of heterotrophic and higher organisms | Physical presence | Clean surfaces | Algae, photosynthetic bacteria, including cyanobacteria |
| Breakdown of material | Hydrolytic enzymes | Wood, painted surfaces | Fungi, bacteria |
| Disaggregation of material | Filamentous growth form | Stone, concrete, mortar, wood | Fungi, actinomycetes, cyanobacteria, algae, lichens |
| Formation of patinas | Oxidation of translocated cations | Stone | Iron and manganese oxidising bacteria; fungi, cyanobacteria |
| Degradation ("Corrosion") | Acid production | Stone, concrete, mortar | Fungi, bacteria, lichens |
| Weakening and dissolution of structure | Mobilisation and chelation of ions | Stone, brick, concrete, mortar | All |
| Alkaline dissolution | Uptake of $H^+$ ions by cells | Stone | Algae, cyanobacteria |
| Disruption of layered silicates | Liberation of polyols (e.g. glycerol, polysaccharides) | Mica, soapstone | All |

Microbial biofilms, complex associations of microscopic organisms and their metabolic products, may be visible or invisible to the naked eye. Almost all surfaces can be so colonized (Koestler et al., 1985; Griffin et al., 1991). "Soiling" and discoloration of buildings is usually evidence of a biofilm, but even invisible biofilms can be a threat to the structure, producing acids and other substances (see Table 1), which can degrade the surfaces of mineral materials and cause spalling (flaking) of surface coatings. The rate of colonization is determined by environmental, as well as biological, factors. Temperature, humidity, light intensity and the physico-chemical nature of the surface of the material all play an important role. Generally, hard, polished stone of low porosity is more resistant to the degradative attack of microorganisms, but all rocks are susceptible to biofilm formation. The various types of stone used in historic constructions in Latin America are shown in Table 2, with examples of the important cultural sites built using these materials.

**Table 2. Rocks used as constructional materials in historic buildings in Latin America (LA)**

| Rock type | Examples | Porosity (%) | Other relevant characteristics | Examples of constructions | Fig. No. |
|---|---|---|---|---|---|
| Siliceous | Quartzite | 1-2 | Very hard, acid | Minas Gerais churches, Brazil. | |
| | Granite/ Gneiss | 1 | Hard, acid | Machu Picchu, Peru. Cuzco cathedral, Peru | 2 |
| | Sandstone | 5-30 | Variable durability, acid | Colonial buildings in LA | |
| | Soapstone | 0.5-5 | Soft, basic, acid-resistant | Minas Gerais churches; Statue of Christ, Rio, Brazil. | 3, 4 |
| | Slate | 0.5-5 | Basic | Quilmes, Argentina; Tiwanako, Bolivia; Colonial buildings in LA. | 5 |
| Limestone | Limestone | 2-20 | Basic | Mayan constructions in the Yucatan peninsula, Mexico; colonial buildings, Cartagena, Colombia. | 1, 6 |
| | Marble | 0.5-2 | Basic | Funerary monuments. | 7 |

Fig. 2. The Inca site of Machu Picchu, Peru.


Fig. 3. The Church of Bom Jesus, Congonhas, Minas Gerais, Brazil. The unpainted part of the church façade is of quartzite and soapstone. The statues of the prophets are carved in soapstone.

Fig. 4. Statue of Christ, Corcovado, Rio de Janeiro, Brazil. The concrete statue is covered with a soapstone mosaic.

**Fig. 5. Skulls set into wall at Tiwanaco, Bolivia. Note orange-yellow lichen on left hand side.**

**Fig. 6. Limestone carving on a Mayan building at Chichen Itza, Mexico.**

**Fig. 7. Marble tombstone with intense black biofilm in churchyard in Minas Gerais.**

Historic buildings have obviously survived for centuries and it may be suggested that they therefore have an inherent resistance to decay processes. Of course, they are normally subject to renovation and repair over the years. However, environmental conditions have changed recently in such a way as to increase the activities of biofilms on mineral surfaces. The effect of industrialization on weathering has been discussed by Winkler (1976). Layers of organic pollutants may act as nutrients for the growth of heterotrophic microorganisms (bacteria and fungi), thus accelerating both aesthetic and physico-chemical deterioration (May et al., 1993; Saiz-Jimenez, 1995). The adsorbed pollutants include fatty acids and aliphatic and aromatic hydrocarbons (Saiz-Jimenez, 1995; Zanardini et al., 2000), which modify the nature of the stone surface. Increased inputs of sulfur and nitrogen, as acid rain, and trace elements from particulates can also stimulate microbial growth.

Coatings such as plaster and paint also modify the stone surface. Paints and varnishes reduce the ingress of water and can be protective, but they also lead to the retention of water once present under the coating, and this accelerates internal degradation. A similar situation can occur beneath patinas, surface layers of inorganic oxides, whose formation can be induced by microorganisms (Krumbein et al., 1987). Painted surfaces are subject to microbial deterioration, the organic constituents of the paint frequently acting as food for the cells.

Salts of organic acids, produced by microbial cells during their normal metabolic processes, can mobilize cations from within the stone, causing degradation (Petersen et al., 1988, Saiz-Jimenez, 1994). It has been suggested that iron-chelating compounds (siderophores) produced by living cells can induce microfissures in the stone surface (Krumbein & Schönborn-Krumbein, 1987).

The majority of information on biodeterioration of ancient buildings comes from research in Europe, even though the processes are more rapid under tropical and sub-tropical conditions. We report here the microorganisms in biofilms on historic buildings in the Latin American countries of Bolivia, Brazil, Colombia, Equador, Mexico and Peru, and their potential for biodeterioration.

## 2 EXPERIMENTAL

Samples of biofilms were taken from the surfaces of various historic buildings in Bolivia, Brazil, Colombia, Equador, Mexico and Peru, using the non-destructive adhesive tape sampling method of Gaylarde & Gaylarde (1998), and fungi and phototrophic microorganisms (algae and cyanobacteria) were identified by culture and microscopic analysis. Fungi were identified both on algal and fungal media (Shirakawa et al., 2001). Bacteria, actinomycetes and protozoa were noted in cultures on algal media, but were only identified by morphology, where possible. In addition, samples of degraded rock from both natural sites and buildings were taken to study the role of microorganisms in fissures and within the structure of the rock (endoliths).

## 3 RESULTS AND DISCUSSION

The major groups of microorganisms detected in the superficial biofilms are cyanobacteria and fungi. There is no significant difference between similar buildings in different countries, but there is a difference in the major types of microorganisms detected on different materials (Table 3), which has also been reported by other authors (Tomaselli et al., 2000). These organisms are able to survive the continual drying and rehydration occurring on exposed building surfaces. Although some, such as the alga, *Trentepohlia*, produce specialized survival cells, the major microbial genera detected, *Gloeocapsa*, *Synechocystis*, *Cladosporium* and *Aureobasidium*, grow in our laboratory cultures from their vegetative forms, collected from the walls on adhesive tape and stored under dry conditions prior to culture. This ability to survive extreme and prolonged desiccation is an essential characteristic of cells which colonize walls exposed to the external environment. Many of the organisms produce pigments for protection against uv and these can cause the staining seen on exposed surfaces of buildings. Algae and cyanobacteria are always intrinsically coloured when active, and generally assume a grey/black coloration when dead. The most prevalent fungi are dark-pigmented types and the actinomycete genus *Geodermatophilus* is also typically pigmented. Thus the principal microorganisms detected on these historic buildings cause aesthetic deterioration of the surface (discoloration). However, they can also actively degrade the materials.

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**Table 3. Main types of organisms detected on the surfaces of historic buildings in Latin America**

| Group | Characteristics of cells detected | Most common occurrence* | Fig. No. |
|---|---|---|---|
| **Pleurocapsales** | Single-celled or colonial cyanobacteria, often darkly pigmented | L, S, M | 8 |
| **Gloeocapsa** | Single-celled or colonial cyanobacteria, encapsulated, cells and/or capsules often coloured pink, purple or brown | L, P | 9 |
| **Synechocystis** | Single-celled or colonial cyanobacteria, sometimes in coloured mucilage | P, S | 9 |
| **Oscillatoriales** | Filamentous cyanobacteria, with or without a sheath, generally green or brown | P | 10 |
| **Scytonemataceae** | Filamentous cyanobacteria, sheathed, often dark brown | P | 11 |
| **Nostocaceae** | Filamentous cyanobacteria, embedded in mucilage, often brown or grey/blue | G, P | 12 |
| ***Actinomycetes*, Streptomyces, Nocardia, Geodermatophilus** | Filamentous bactéria. *Geodermatophilus* frequently pink or brown coloured | All | 13 |
| **Cladosporium** | Dark pigmented filamentous fungus | All | 14 |
| **Aureobasidium** | "Black yeast" | P | |
| **Trentepohlia** | Filamentous green alga, often orange, brown, or pink in mass | P, G | 15 |
| **Chlorella, Chlorococcum** | Coccoid green algae | P | |

***L (limestone), S (soapstone), G (granite), M (mortar), P (painted or otherwise coated) surfaces**

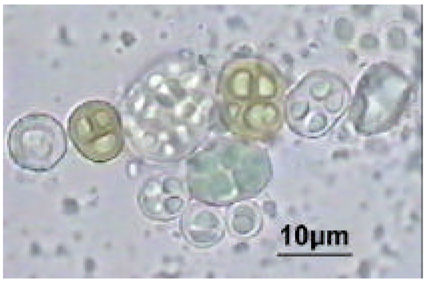


**Fig. 8. Pleurocapsales group**

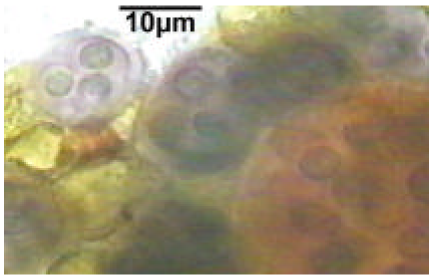


**Fig. 9. *Gloeocapsa* (purple) and S*ynechocystis* genera**

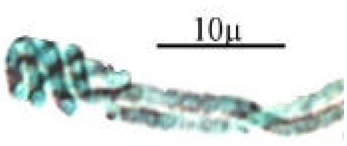


**Fig. 10 Oscillatoriales group.**

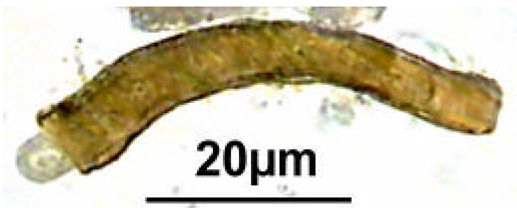


**Fig. 11. Scytonemataceae group**

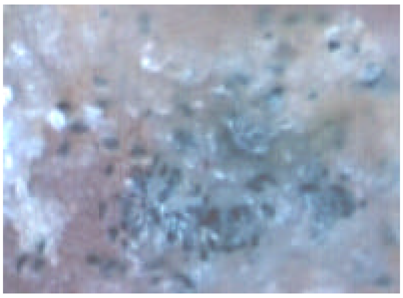

**Fig. 12. Colonies of the cyanobacterial group *Nostocaceae*, growing endolithically in a granite church in Parati, Brazil.**

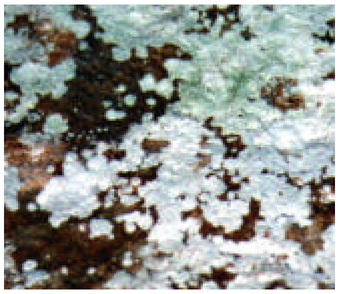

**Fig. 13. Actinomycetes growing as a white layer on an internal wall at the Mayan site of Uxmal, Mexico.**

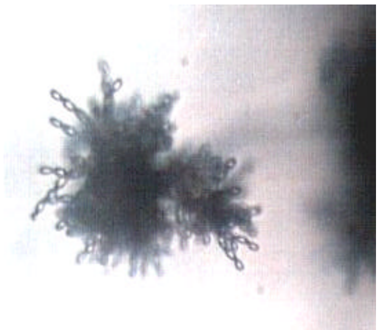

**Fig. 14. The filamentous fungus, *Cladosporium*. The spore-bearing head is shown here.**


**Fig. 15. The filamentous alga *Trentepohlia*, showing brown/red oil droplets inside cells.**

**Degradation of siliceous minerals.**

The ability of the microorganisms found on these surfaces to survive desiccation indicates that they produce osmotic protectants, polyols, which partially replace water in the cell cytoplasm. Glycerol has long been known to cause the expansion of micaceous minerals. Glycerol and other polyols, such as glucose and high molecular weight polysaccharides, form complexes by hydrogen bonding in between the laminar polysiloxane crystal planes of micaceous minerals. This causes expansion and mechanical stresses, which may degrade the rock, and ions bound within the structure may become more accessible to chemical attack. The polyols present in high concentrations in desiccation-resistant organisms such as those found on the historic buildings in this study can thus be responsible for weakening of the structure of siliceous materials. These chemicals protect the cells against freezing, desiccation and excess salt. Our studies show that *Gloeocapsa*, *Scytonema*, *Aureobasidium* and *Cladosporium* from paint will grow on media with up to 20% (w/v) added NaCl. Unpublished observations on samples from mortar collected at a site with severe salting in Europe, showed a much wider range of salt tolerant genera, including slime moulds, protozoa and rotifers. The role of polyols in biodegradation of siliceous materials has not previously been suggested in the literature.

It is commonly stated that siliceous rocks are degraded by biologically produced acids (Strzelczyk, 1981; Petersen et al., 1988). However, Bennett et al. (1988) showed that organic acids, in general, do not increase the dissolution of silica. In the presence only of citrate, salicylate, or oxalate anions, the dissolution increased with pH over the tested pH range of 3 to 7. Anions of α-dicarboxylic acids (e.g. citrate) will form cyclic hydrogen bonded complexes with the superficial hydroxyl groups of quartz. The dissolution of silica will be accelerated by increased hydroxyl ion concentrations, which attack the Si-O-Si linkage and this reaction will almost certainly be favoured by the presence of polyols, which will additionally increase the solubility of these products. Thus polyols and complex organic anions can attack siliceous minerals under alkaline conditions. Silica dissolution under these conditions will also be increased by the formation of covalent siloxanes and hydrogen-bonded complexes with α-hydroxy acids (eg salicylate) and α-phenolic diols (eg humic acids, pyrocatechol), all of which are produced by microbial activity (Iler, 1979).

The siliceous mineral, soapstone, is a soft stone that is very resistant to acid and has been used to construct tanks for the storage of strong mineral acids. Its main component is talc, $Mg_3Si_4O_{10}(OH)_2$. This stone has been extensively used in the historical monuments of Minas Gerais, Brazil (Fig. 2). It has an alkaline reaction, generally pH 9 or a little higher. Silicon and aluminium are mobilized at pH values above pH 9·5 and phototrophs can increase their immediate environment to above pH 11 by the action of ionic pumps (Miller et al. 1990). The resulting stone degradation can produce catastrophic sloughing of surface layers when the organisms are present within the material as endoliths. This is shown in Fig. 16, where microorganisms growing within the soapstone nfaçade of a church in Ouro Preto, Brazil, have caused spalling of the surface. The principal microorganisms detected in the deeper part the soapstone sample were the cyanobacterial genus *Synechococcus* and actinomycetesnn.

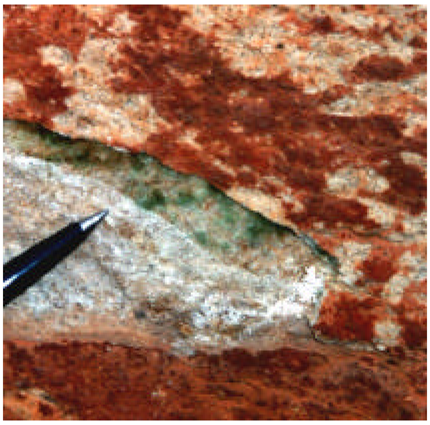

**Fig. 16. Spalling soapstone, showing green growth of (mainly) cyanobacteria deeper than the whitened spalled layer. The red coloration is patina.**

**Degradation of carbonates.**

Limestone may be biodegraded by bacterial, fungal and algal acids (Koestler et al., 1985 Grant, 1982; May et al, 1993) and by mechanical penetration by filamentous fungi and phototrophs (Hoffmann, 1989; Ortega-Calvo et al., 1991). Dark pigmented mitosporic fungi, or "black yeasts", can also actively penetrate limestone, causing "biopitting" (Sterflinger & Krumbein, 1997).

Hoffmann (1989), in his review of algae in terrestrial habitats, states that cyanobacteria capable of boring into limestone are of the genera *Gloeocapsa*, *Stigonema*, *Chroococcus*, *Aphanocapsa* and *Schizothrix*.. All of these genera, apart from *Schizothrix*, are found on the historic buildings in our study.

The mechanism of boring is unknown. Cyanobacterial cells on limestone can often be seen, in electron micrographs, to be covered with calcareous deposits (Ascaso et al., 1998; Ortega-Morales et al., 2000), which suggests the migration of calcium from neighboring sites. Phototrophs deposit $CaCO_3$ in the light and solubilize it at night because of changing bicarbonate concentrations. This process has been well studied in the coccoid cyanobacterium *Synechococcus* GL24. These spherical cells possess on their external surface a so-called S-layer, which binds calcium ions at negatively charged sites (Schultze et al., 1994). The bound calcium complexes with carbonate ions at pH values above 8.3. Even if the pH of the stone surface is not so high, it is elevated to these levels by phototrophic activity, whereby $OH^-$ ions are released and concentrated around the cells (Miller et al., 1990), raising the local pH. Cells of *Synechococcus* can become encrusted with calcite within 8h in a suitable environment and must continually shed patches of mineralized S-layer to remain viable (Douglas & Beveridge, 1998). Mobilization of calcium ions by such metabolic activity and ion transport, in addition to the trapping of released particles of calcite, either of biotic or abiotic origen, in the gelatinous cyanobacterial sheath (Pentecost, 1988), is an important mechanism of limestone degradation by cyanobacteria and algae.

## 4 CONCLUSIONS

Pigmented microorganisms, bacteria, fungi and algae, cause discoloration on the surface of buildings of historic and cultural importance. In addition, they can directly cause degradation of the materials through various metabolic activities. This biodeterioration and biodegradation is not prevented by surface coatings on stone buildings, since these coatings, themselves, are subject to microbial growth. Although hard, smooth and less porous surfaces, such as basalt or varnished stone, are more resistant to microbial colonization, they can still be attacked by many microorganisms. The resultant biofilms should be removed regularly, using non-abrasive and environmentally safe methods, to reduce the impact of microbial activities.

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# Cathodic Protection Of Steel Framed Heritage Structures

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**Summary: The corrosion of early 20th century steel-framed structures is resulting in serious damage to the integrity and appearance of many historically important structures. Conventional repair technologies are expensive and disruptive and can result in sensitive buildings becoming little more than modern reproductions. Furthermore, traditional repair methods do not necessarily provide a sufficient extension of life.**

**Historically cathodic protection may be seen as the first engineered solution to the problems of ferrous alloy corrosion. Recent improvement in anode design and control and monitoring hardware have made the technique available for use with steel framed structures in a manner that is acceptable both commercially and from a conservation viewpoint.**

**This paper discusses the practicalities of steel frame cathodic protection and the general approaches available to achieving it. It will also cover the current research to establish more precisely many of the design and operational characteristics of the technique.**

**Keywords: Steel framed, heritage, corrosion, cathodic protection.**

## 1 INTRODUCTION

The form of steel frame building construction, initially employed in Chicago and subsequently used in most major western cities in the first two decades of the 20th century, has resulted in serious consequences with respect to serviceability, safety and aesthetics. Most notably, the identification of "Regent Street Disease" in the United Kingdom in the late 1970's first highlighted the problems of steel-framed corrosion. Many of the grand, high profile, and often-protected structures, in the centres of many cities have been affected (Gibbs 2001).

While the 'modern' problem of steel frame corrosion dates back less than 25 years, the problem was originally encountered and recognised over 50 years ago, quote:

"One interesting case of corrosion in a steel-framed building was investigated in collaboration with the Chemical Research Laboratory. Extensive corrosion of the steelwork had caused cracking of the external walls. The photograph (reproduced in Figure 1) shows a layer of rust up to half an inch thick on a truss member. The frame was generally encased in brickwork bedded in a black clinker mortar, clad with either glazed brickwork or Portland stone. It was concluded that the corrosion of the steelwork was due primarily to deficiencies in design which had allowed water to gain access to the steel, aggravated perhaps by the use of a clinker mortar, by the presence of soluble salts in the brickwork, and by inadequate painting of the steel". (Department of Scientific and Industrial Research 1947).

The problems observed form part of a pattern of decay that has only recently been formally recognised and one that is expected to become more apparent over the next decade. It is a direct consequence of the age and nature of construction.

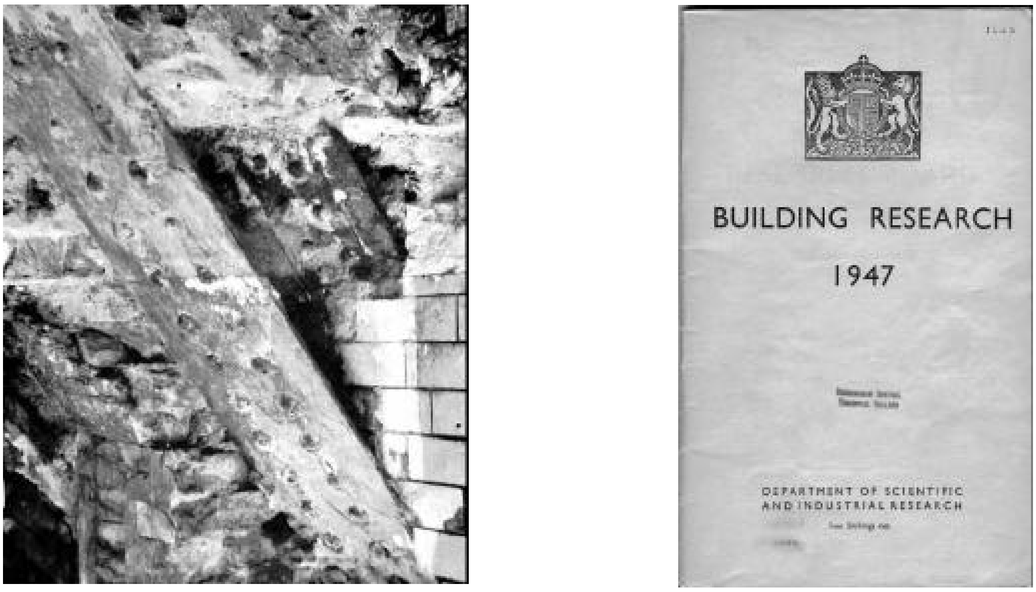


**Figure 1. Early 20th C steel frame showing corrosion, as reported in 1947.**

Cathodic protection, originally developed by Humphrey Davy and later employed widely on buried and submerged structures, was first considered for reinforced concrete in the late 1950's. It became a serious commercial solution after the development of improved anode systems in the early 1980's. The transfer to steel-framed buildings was somewhat slower and it was not until 1997 that the first sizeable structure was protected by such a system (Figure 2) (Evans 1997).

## 2 CORROSION OF STEEL

In the presence of moisture and oxygen, steel rusts. The rate and nature of the process depends on alloy composition, environmental factors, design and nature of additional protection but on average one tonne of steel is lost every 90 seconds in the UK as a direct consequence of corrosion.


**Figure 2. Gloucester Road Underground Station, London.**

In its simplest form the corrosion process can be represented by two dissimilar metals in an aqueous electrolyte, joined to allow electrons to pass from anode to cathode. In reality, when a metal corrodes, anodic and cathodic areas can be formed on a single surface in contact with the aggressive aqueous environment. As a result, corrosion can occur at a large number of sites over the surface of the metal. Dissolved metal ions react with hydroxyl ions to form corrosion products (Lambert 2001).

The relative humidity of an environment has a profound effect on the rate of corrosion of steel. There is a critical level of relative humidity below which corrosion does not occur and often secondary and tertiary levels above that the corrosion rate increases significantly. In the case of steel, corrosion commences at a slow rate at approximately 60% RH, the rate increases at 75-80% RH and again at 90%. Contamination of the environment has a tendency to reduce the relative humidity at which corrosion is initiated (e.g. the presence of salts) (Vernon 1935).

Controlling the relative humidity of encased steel and reinforced concrete can provide an effective means of controlling reinforcement corrosion, particularly where the removal or exclusion of excess moisture also removes or prevents the ingress of potentially aggressive species. As most of the moisture and other mobile species that influence durability must cross the boundary between substrate and atmosphere, the application of coatings and surface treatments can be highly effective at limiting or preventing degradation, subject to aesthetic and heritage considerations (Lambert 1997).

### 2.1 Steel frame corrosion

A pattern of corrosion-induced damage is now being widely observed in steel-framed structures, typically constructed pre-1930's (Jones *et al.* 1999). The mechanism of the damage can be summarised as follows and is illustrated in Figure 3:

- The steel frame needs to be protected from its natural tendency to corrode (i.e. return to a more stable condition, rust, through an electrochemical reaction in the presence of moisture and oxygen).

  At the time of construction the protection typically consisted of little more than a cement wash or thin bituminous coating followed by partial encasement in concrete or mortar. While concrete encapsulation can provide excellent long-term protection to steel as both a physical and chemical barrier, the original coating would not be sufficient to prevent corrosion in the presence of sustained high levels of moisture.

- The gradual breakdown of joints, pointing and flashing increasingly allow water ingress. As expansive corrosion products are formed brick or stone cladding can be displaced, further opening up joints and cracks and permitting greater access to water. Thus, the rate of degradation will tend to accelerate.

  Thermal movements that aggravate the opening of joints will also lead to an acceleration of the damage, as typically observed on the weather-exposed corners of such buildings.

The rate at which the damage to the cladding occurs is governed by a number of factors:

- The time at which corrosion initiates – largely dependent upon location, aspect and level of previous maintenance.
- The rate at which corrosion progresses – largely dependent upon availability to moisture and oxygen.
- The intimacy of the contact between the corroding steel and the cladding – gaps between steel and cladding can accommodate extensive corrosion with no visible damage.

**Figure 3. Examples of steel-frame corrosion.**

Where the steel is surrounded by a gap, the risk of displacing the masonry cladding is greatly reduced although the likelihood of suffering significant loss of section is much higher, particularly in the upper levels of buildings where exposure conditions are generally more severe. In fact the location and severity of damage on a particular building can often be seen to follow a particular pattern, as illustrated in Figure 4.

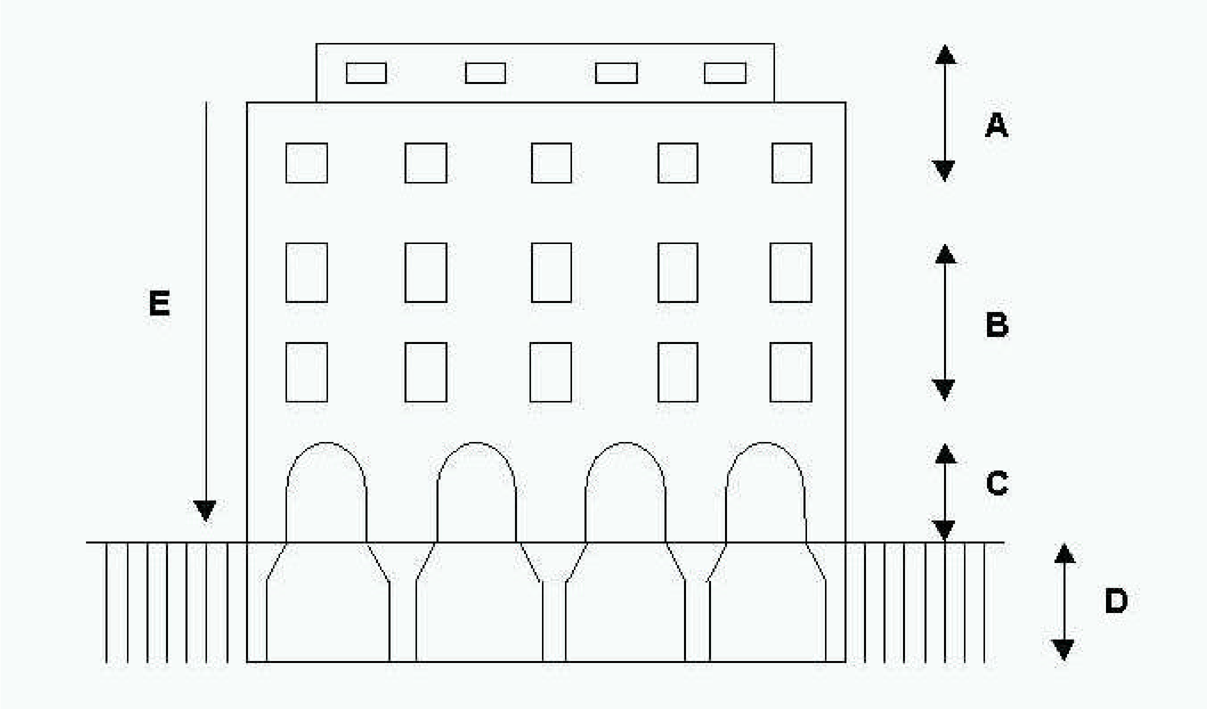


**Figure 4. Typical Pattern of Damage Distribution**

**2.2 Area A: Upper levels, including penthouse.**

Damage is often most severe in this location, aggravated by degraded or inadequate roof coverings and rainwater goods. The top levels of such buildings are often both elaborately decorated and grossly under-maintained with the consequence that the risk of displaced and falling masonry can be very high. Considerable removal and reinstatement of damaged material can be required in such areas.

**2.3 Area B: Middle levels.**

The mid-band of steel-framed buildings often display only moderate levels of corrosion. Left untreated, the corrosion will eventually progress to the stage where disruption of the masonry cladding occurs.

**2.4 Area C: Ground level.**

In general, the ground levels have little or no serious damage. Not only are such areas often more sheltered but are also subject to the highest levels of continual maintenance with problems quickly and easily identified and repaired. The masonry at ground level is also often heavier with a superior quality of construction and this no doubt also contributes to the reduced risk of damage.

**2.5 Area D: Basement.**

The level of damage associated with basements can often be quite high. This may be due to a number of factors including inadequate tanking leading to groundwater ingress and the proximity to de-iced pavements and roadways. The damage associated with the leakage of rainwater and other drainage pipes is also often focussed on the basement level.

**2.6 Area E: Exposed face or corner.**

Not all cracks in steel-framed buildings are initiated by corrosion, though most encourage its progression. Thermal movements can cause the progressive jacking open of joints allowing corrosion to initiate and proceed. The facade facing the sun and exposed to the prevailing wind-blown rain will suffer preferentially. Corners, irrespective of orientation, will generally suffer more than mid-facade. Where the corners of building have suffered thermal crack and associated corrosion, considerable traditional repair may be required.

Awareness of such a damage pattern can be valuable when developing the inspection of steel-framed buildings and in particular, helping to target any intrusive investigation of such structures. The pattern described above is largely based on UK experience and therefore generally relates to structures of less than 10 stories exposed to a temperate environment.

**Table 1. Repair Options for Steel Frame Corrosion**

| *Remediation Option* | *Description* | *Considerations* |
|---|---|---|
| Do nothing /monitor. | Carry out minimum repairs and monitor the continuing degradation until further action is required. This may involve the use of embedded corrosion sensors. | Such an approach is appropriate for those areas that have the potential for corrosion but are presently not actively corroding, e.g. Areas C or D in Figure 4. |
| Conventional repair. | Repair areas where steelwork has suffered significant loss of section and areas where expansive corrosion has resulted in significant disruption to the adjacent building fabric. | Reconstruction is the most effective long-term solution but is disruptive and expensive and hence should be restricted to localised areas that are considered essential. Typically appropriate for Area A. |
| Corrosion inhibitor. | Inhibitors, usually based on amino alcohols, can be applied to exposed surfaces, injected, buried as emitters or fogged into voids to control corrosion of the steelwork. | Corrosion monitoring is recommended to ascertain effectiveness of the inhibitor and reapplication would be anticipated at 5-10 year intervals. Most appropriate for Area C. |
| Cathodic protection. | Steelwork is protected from corrosion by the application of a small current at low voltage.<br><br>The current is provided by anodes inserted into the mortar infill between the cladding and the steel frame or between joints in the masonry. | On-going monitoring and adjustment is required. Time to first maintenance is determined by the life of the anodes that should provide a minimum of 25 years service. Appropriate for areas B, C and D. |

Changes in building height, environment and local methods of construction are known to influence this basic pattern and must be taken into consideration when carrying out inspections and developing repair solutions.

## 3 REPAIR OPTIONS

A number of remediation options are applicable to treat the range of conditions observed on steel-framed structures suffering from vary degrees of corrosion-related damage, most notably they include the four approaches outlined in Table 1.

While all such approaches to repair are valid and employed as appropriate, cathodic protection may be seen to have particular advantages with respect to the preservation of historically significant structures, combining both long life and minimum disruption to the original structure.

## 4 CATHODIC PROTECTION

Although the beneficial effects of cathodic protection have been recognised since the middle of the eighteenth century, it is only during the second half of this century that the technique has been seriously employed, predominantly in the protection of pipelines, ships and oilfield structures. More recently, the technology has been refined and applied for the protection of structural steel particularly that embedded in concrete but equally well for other steel elements encased in mortar, plaster or masonry.

The systems employed for steel-framed buildings have been developed from the extensive experience gained in the cathodic protection of reinforced concrete (Chess, 1998).

Corrosion of steel, being an electrochemical process, results in the formation of anodic and cathodic sites on the surface of the steel. Under typical atmospheric conditions metal is dissolved at the anodic sites while the cathodic areas remain unaffected. By applying a small externally generated current to the steel it is possible to make all the steel cathodic and therefore non-corroding.

The externally applied current can either be produced by a material that will corrode preferentially to the steel - a ‘sacrificial’ anode such as zinc, or provided by a low voltage DC source via an effectively inert material to provide an impressed current to the steel.

Impressed current systems are driven by the application of a direct current through an inert or effectively inert anode. The potential of the reinforcement is depressed by increasing the applied current, which is generally supplied from the mains using a transformer/rectifier to provide a direct current supply. Ideally the potential should be depressed to a level where corrosion is not thermodynamically possible, but any reduction in potential will lead to a reduction in corrosion rate.

Cathodic protection can be applied to any structure where the steel is in continuous contact with concrete or mortar encasement, the pore solution of which acts as an electrolyte. If the steel is not continuous then local anodic and cathodic sites may be developed under the influence of the impressed current, leading to stray current corrosion. Where electrical discontinuity is found, or suspected, bonding or connection by cable can be provided to ensure electrical continuity throughout.

Hydroxyl ions are produced at the cathode (i.e. reinforcement) which increase the alkalinity. There is a slight possibility that this increase in alkalinity may initiate alkali-aggregate reaction in susceptible aggregates, although this effect has not been reported in any protected structures.

Hydrogen gas may be produced at the cathode if the potential is sufficient for electrolysis of water (electrolyte) to occur. The steel/concrete potential must therefore be carefully monitored. Hydrogen evolution can cause embrittlement of highly stressed steel and for this reason prestressed or post-tensioned reinforced concrete structures are generally not protected by cathodic protection in the UK, although cathodic protection of pre-stressed structures is undertaken in Italy.

Impressed current cathodic protection systems require regular monitoring since the current requirements for the system may vary as a result of many factors including variations in resistivity of the concrete due to variation in moisture content, changes in the environment around the reinforcement as a result of the applied current, etc.

Cathodic protection systems must be carefully designed and account must be taken of many different factors such as the aggressiveness of the environment; the area of steel to be protected; the resistivity of the surrounding material; the positioning of any external metallic objects which could be affected by the system and the type of anode employed.

### 4.1 Design

Conventional cathodic protection design is based on calculating the area of steel to be protected and selecting an appropriate current density. A suitable anode system can then be selected based on various site considerations such as access, environment, and the required current demand.

Cathodic protection design for steel frame buildings has a different emphasis with the primary concern being disruption to the façade of the structure. Anode systems are selected based on these criteria. Achieving adequate current distribution is the next important consideration. Due to the variable nature of the fill material surrounding the steelwork this is often best established by carrying out a pilot installation over a small section of the building, typically including a length of beam and column.

In addition to allowing the anode type and spacing to be optimised, a pilot installation provides the opportunity to establish the aesthetic impact of the installation. This proves particularly beneficial where the structure is subject to statutory local or national government approval prior to installation by allowing relevant organisations to inspect a sample of the work and observe the method of installation.

### 4.2 Selection of anode systems

There are two basic systems that are in use for cathodic protection installations of this type, discrete anodes based on titanium oxide ceramic or titanium and expanded titanium ribbon anodes. Where titanium metal is employed, the surface must be coated with a mixture of metal oxides to prevent the titanium anodising.

The discrete anodes are typically much smaller than those used in reinforced concrete to minimise the aesthetic influence of the installation and to enable a more even current distribution. The ribbon anodes have been employed for many years in cathodic protection systems for reinforced concrete either in combination with other materials or in their own right. (Atkins & Davies 2001)

The majority of cathodic protection systems installed on steel-framed buildings to date have been based on discrete anodes. This is due to the ease of installation and adaptability of such a system. However ribbon anodes do provide a suitable option if it is possible to gain access to continuous strips of mortar, for example if there is an appropriate void within the building that provides such direct access to the infill, or if large lengths of the frame are being exposed and refilled with mortar during the repair process.

### 4.3 Installation

The installation process for both systems is relatively straightforward and does not necessarily require the use of a specialist repair contractor. If the system is to be installed from the exterior of the structure the bulk of the work involves cutting fine chases for cabling and drilling small diameter holes for the anodes and monitoring probes.

In order to achieve the required aesthetic finish the chases and holes are usually back filled with a material appropriate for the cathodic protection system to 5mm of the finished surface level. The final pointing may then be undertaken using a specialist colour matched material to achieve the desired aesthetic finish.

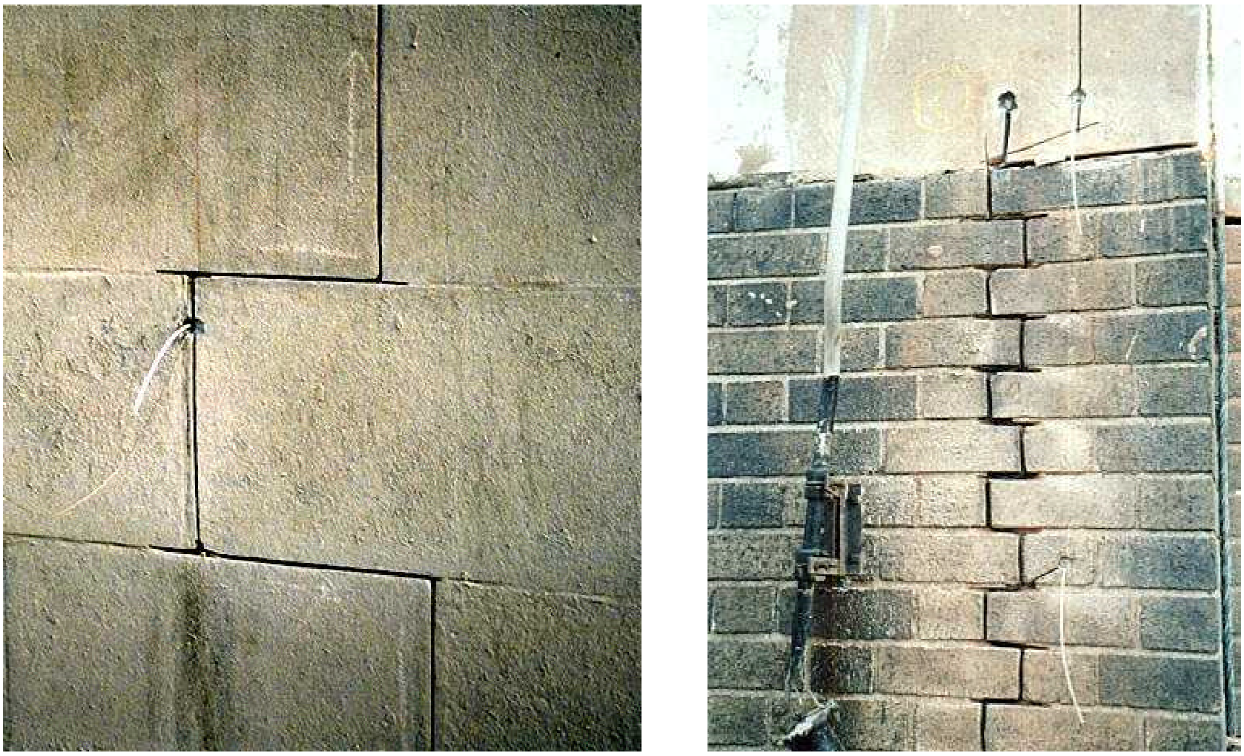

**Figure 5. Installation of ceramic discrete anodes.**

### 4.4 Power, monitoring and control

System monitoring is important with all forms of cathodic protection and this is equally true for steel frame applications. Fortunately, improvements in data handling, manipulation and transmission mean that effective monitoring can be performed relatively easily, even with large and complex installations.

The development of smaller and more integrated power, monitoring and control systems have played a vital role in extending cathodic protection solutions to building structures by employing many of the latest developments in digital technology and internet-based communications.

Particular considerations for steel-framed structures include limiting the size of power and monitoring enclosures and the extent of cabling. In both cases, order of magnitude reductions have been possible, allowing installation to proceed without disrupting the operation of the building or altering the outwards appearance.

### 4.5 Protection criteria

There are a number of protection criteria available in international standards for Cathodic Protection. These are generally based on empirical experience, e.g. 100mV decay in 24 hours (British Standards Institution, 2000), or theoretical considerations that can be based on inappropriate assumptions e.g. a potential of –600mV vs. Standard Hydrogen Electrode (Pourbaix, 1974). For the purposes of steel framed buildings the former is more appropriate, although there is little formal guidance on the suitability of this or other criteria.

### 4.6 Stray current

The issue of stray current corrosion in cathodic protection systems is often a concern. In reinforced concrete systems for example, bars are rarely welded together and so electrically discontinuous steel can often be encountered. If this is not remedied the isolated reinforcement can be subject to stray current corrosion where the cathodic protection system drives current through the discontinuous steel leading to accelerated corrosion where the current is discharging. Typically for reinforced concrete systems, continuity between reinforcement bars is investigated during the installation phase to ensure all the reinforcement is electrically continuous.

For steel framed structures, electrical continuity between structural members is rarely a problem, since the structural connections are typically bolted or riveted. However, there are a number of items such as metal window frames or drainage downspouts that are invariably electrically discontinuous these must be considered during the site phase of the works. If the items are connected to earth, as would be expected for any electrical installation, e.g. lighting brackets, the earthing system prevents stray current effects.

On historic structures the earthing requirements may not be in accordance with present standards and so the possible effects of this must be assessed and appropriate remedial actions undertaken. Typically this involves either electrical isolation from the surrounding material, possibly by replacing fixings with a resin-anchored type, or by bonding the discontinuous items into the system. Alternatively, it may be sufficient to employ monitoring during commissioning and carry out remedial isolation or bonding if required.

## 5 DEVELOPMENT OF DESIGN GUIDANCE

In order to properly quantify many of the factors associated with the design, installation and long-term operation of impressed current cathodic protection systems for steel framed structures, a three year research project has recently commenced at Sheffield Hallam University in the UK.

One of the major problems in understanding the mechanisms of cathodic protection in steel-framed construction is the relatively complex geometry of the system under consideration. No formal information exists with respect to current throw onto typical steel sections yet this is fundamental to the design of the systems.

Initial studies are being carried out on a range of steel and anode geometries employing a sandbox to represent the surrounding masonry. This technique has previous been employed to study the throw of current from ground-beds to pipeline sections but is not believed to have been previously used in this context. This technique also allows the risk and magnitude of stray current effects on discontinuous metallic components, e.g. cramps and wall-ties, to be formally evaluated for the first time.

On completion of the sandbox work, a number of geometries will be selected for further testing with mortar and brick encasement. From field experience it is estimated that such specimens would not need to be more than 1 to 1.5 m in length to facilitate manufacture and manipulation within the laboratory environment. The suitability of zinc-based sacrificial systems is also to be assessed for specific applications where an impressed system is considered overly complex or otherwise inappropriate.

In parallel with the laboratory work, a study of existing installations is programmed to be carried out and discussions or questionnaires organised with the various bodies associated with the repair and preservation of such structures. It is hoped that it will be possible to communicate with overseas bodies elsewhere in the world where the problem of steel frame corrosion is also a concern but no cathodic protection installations have yet been carried out.

From this study it should be possible to generate proper, well-founded guidance on the design and operation of cathodic protection systems for such sensitive and important applications.

## 6 ACKNOWLEDGMENTS

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# Use Of Additivated Lime Mortars For Old Building Rehabilitation Adapted Testing Methods

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**Summary: The use of natural pozzolans as additives in lime mortars is a possibility that permits the execution of mortars compatible with old renders and masonry. On the other hand, these mortars provide better mechanical properties and it is believed that they possess an increased durability, in comparison with lime mortars. Recently developed European Standards try to fill the gap of standardization in this field, but some difficulties are experimented in their application. In the present paper European Standard EN 1015-11 is analysed in terms of preparation and conditioning of mortar specimens. Further preparations and conditionings are tested in order to verify their adequacy for use for additivated lime mortars. Results of mechanical, physical and chemical tests are presented and discussed.**

**Keywords. Mortar, Lime, Pozzolans, Testing Methods, Curing Conditions**

## 1 INTRODUCTION -USE OF NATURAL POZZOLANS IN LIME MORTARS

### 1.1 Brief historic perspective and present experience

The rehabilitation of old buildings comprises the maintenance and/or repair of old renders, for which mortars presenting compatible characteristics with adjacent masonry and remaining renders should be used. The main component of old renders is usually lime, occasionally enriched by the presence of pozzolanic or other additives. The use of natural pozzolans is well known, having been mentioned by Vitruvio as a material provenient from the region of Pozzuoli and been included in the preparation of mortars and concretes throughout the times (Illston, 1994) as they confer hydraulic properties and therefore enable hardening under water or in wet conditions. It is also believed that the addition of pozzolans confers greater durability to lime mortars, permitting to overcome the need for permanent maintenance.

In Portugal, this natural material of volcanic origin can be found in the Azores islands. In previous studies developed at LNEC (Velosa and Veiga, 2001), Azores pozzolans and Cape Verde pozzolans, the latter of which is commercialised and currently used in Portugal, were used as additives in lime mortars. Comparative physical, mechanical and chemical tests were undertaken in order to determine the effects caused by the addition of these pozzolans; results proved the adequacy of these additivated mortars for use in conservation and the significant improvement in mechanical characteristics induced by the addition of Cape Verde pozzolans. In consequence of these results, further studies on the additivation of lime mortars with natural pozzolanic additives seemed both an interesting and promising theme to proceed with. However, the specificity of mortars both with lime and pozzolans requires a study of hardening reactions and the adaptation of present standards so as to guarantee the most adequate curing conditions that will permit to obtain the most relevant results.

### 1.2 Chemical reactions during the hardening process

Limestone is the prime material for the manufacture of lime putty or hydrated lime; after burning at a temperature surrounding 900ºC and subsequently adding water, dry hydrated lime or lime putty will be produced, depending on the quantity of water added. In this study commercial dry hydrated lime was used. Reactions leading to the formation of this product are as follows:

$CaCO_3$ (calcium carbonate) + heat (900ºC) $\rightarrow$ CaO (calcium oxide) + $CO_2$ (carbon dioxide)

CaO (calcium oxide) + $H_2O$ (water) $\rightarrow$ $Ca(OH)_2$ (calcium hydroxide)

In lime mortars hardening will take place through the reaction of calcium hydroxide with carbon dioxide present in the air, forming calcium carbonate as shown in the reaction below.

$Ca(OH)_2$ (calcium hydroxide) + $CO_2$ (carbon dioxide) $\rightarrow$ $CaCO_3$ (calcium carbonate) + $H_2O$ (water)

The main components of pozzolanic materials are silica ($SiO_2$), usually represented as S, and alumina ($Al_2O_3$), usually represented as A; other compounds that are usually present in significantly lower percentages are iron oxide ($Fe_2O_3$), magnesium oxide (MgO) and lime (CaO).

The siliceous compounds present in pozzolanic materials react with the calcium hydroxide from lime and produce calcium silicates, mainly in the form of hydrated bicalcic silicaluminate ($2CaO.Al_2O_3.SiO_2.8H_2O$), hydrated calcium silicates and hydrated calcium aluminates (Coutinho, 1997), products which are responsible for the improvement in durability properties in these mortars. The reactions leading to the formation of these compounds occur in the presence of water and depend on the reactivity and fineness of the pozzolanic materials.

## 2 PREPARATION AND CURING CONDITIONS - STANDARDIZATION

Formerly, European Standards regulated testing methods mainly for cement-based mortars that, since the discovery of Portland cement by Joseph Aspdin in 1824, gradually substituted lime mortars in the execution of renders. Years of experience and studies on the use of cement-based mortars now provide a basis for the belief that they may not be the most adequate for use in renders for old buildings (Torraca, 1996). For conservation purposes, lime mortars and additivated lime mortars fill compatibility requirements in mechanical, physical, chemical and aesthetic terms to a much higher degree (Veiga, 1998).Due to the growing practice, spread throughout Europe, of conserving old buildings rather than building new, the necessity for research and regulations in this area became a reality in recent times. For the specific area of rendering mortars, European Standards EN 1015-1 to EN 1015-4, EN 1015-6 to EN 1015-7, EN 1015-9 to EN 1015-12, EN 1015-17 and EN 1015-19 lay down testing methods, covering various mechanical, chemical and physical testing procedures. For these European Standards, rendering mortars are divided into the following categories:

- Air lime mortars (L)
- Air-lime/cement mortars with cement mass not exceeding 50% of the total binder mass (L/C)
- Cement and air-lime/cement mortars with mass of air-lime not exceeding 50% of the total binder mass (C)
- Mortars with other hydraulic binders (H)
- Retarded mortars (R)

Preparation and curing conditions are described in detail in EN 1015-11, and the procedures are as follows:

**Table 1 – Preparation and conditioning of storing specimens (based on EN 1015-11)**

| Type of mortar | Preparation | Storage time at a temperature of 20 ºC ± 2 ºC in days | | |
|---|---|---|---|---|
| | | Relative humidity | | |
| | | 95 ºC ± 5 ºC or in polyethylene bag | | 65 ºC ± 5 ºC |
| | | in the mould | with the mould removed | with the mould removed |
| L; L/C | 2 | 5 | 2 | 21 |
| C; H | 1 | 2 | 5 | 21 |
| R | 1 | 5 | 2 | 21 |

Preparation 1

- Filling of mould in two equal layers, compacting each layer with 25 strokes of the tamper
- Skimming of excess mortar with a palette knife

Preparation 2

- Placement of mould over glass plate on which two layers of dry white cotton gauze have been placed
- Filling of mould in two equal layers, compacting each layer with 25 strokes of the tamper
- Skimming of excess mortar with a palette knife
- Placement of two layers of white cotton gauze tightly on the mortar surface
- Placement of six layers of absorbent filter paper on top of the gauze

- Coverage with glass plate and turning around of the mould
- Removal of glass plate from the top of the mould and placement of six layers of absorbent filter
- paper on top of the gauze
- Coverage with glass plate and re-inversion of mould
- Placement of 5 kg mass on top

Whilst 'Preparation 1' maintains hydraulic mortars in a humid environment, permitting their hardening by hydraulic reaction, 'Preparation 2' implies the absorption of excess water by the gauze and absorbent filter paper placed on both sides of the mould. However, in this last case, humidity is still maintained during the first seven days. Comparison of results between these distinct types of mortars proves difficult due to the necessity of different curing conditions to assure a rapid hardening. This difficulty is stressed in mortars that harden both by carbonation and hydraulic reaction, which is the case of lime mortars additivated with natural pozzolans.

However, the application as renders requires knowledge of real conditions, as carbon dioxide is always present and relative humidity varies during the day and seasons.

Under atmospheric conditions, mortar behaviour may vary considerably from that conditioned in the laboratory, both in terms of hardening time and properties; it is therefore necessary to verify the differences induced by changes in conditioning.

### 2.1 EN 1015-11 in practice

Formerly, at LNEC, 'Cahier 2669-4' of CSTB was the standard used as the basis for most testing procedures for lime mortars; however, the approval of European Standards led to the option of experiencing their application, namely the specimen execution according to the recent EN 1015-11. Practice revealed the following difficulties:

- Manoeuvring of moulds for 'Preparation 2'.
- Difficulty of mortar to harden in the mould in the specified time and conditions.

In fact, the turning around of a mould covered with a glass plate proves difficult, even to experienced hands, as does the re-inversion of the mould with glass plates on each side. On the other hand, it was verified that mortars did not harden sufficiently for removal from moulds in the time and conditions specified in EN 1015-11 – specimens thus conditioned easily broke or disintegrated upon removal from moulds.

Besides, high humidity during setting is a favourable conditioning for hydraulic mortars but a very unfavourable condition for lime mortars, so the results obtained in such condition will not be representative.

## 3 MATERIALS AND APPLICATION CONDITIONS

### 3.1 Mortar composition

This work is inserted in a larger study where several other compositions were tested

The mortar used for this set of tests was a lime mortar composed of commercial dry hydrated lime (L), Cape Verde pozzolans (P), grinded and passed through the 0.500 sieve, and river sand (S). The volumetric ratio was 1: 0.5: 2.5 (L:P:S). This composition was chosen as it proved to give best results in comparison with other ratios in previous work (Velosa and Veiga, 2001).

### 3.2 Preparation and conditioning – a different approach

Taking into account the difficulties found whilst preparing and conditioning mortars following standards and the specific needs of additivated lime mortars, different conditioning was used, taking into account easiness of practice and relation with real atmospheric conditions.

A larger set of tests is still being developed.

For this work, three different conditionings were adopted:

- Control (C 1)- 23ºC ± 2ºC (temperature); 50% ± 5% (relative humidity), in mould for 7 days
- Filter paper (C 2) - 23ºC ± 2ºC (temperature); 50% ± 5% (relative humidity), with a filter paper on both sides of the mould during 7 days
- Water spraying (C 3) - 23ºC ± 2ºC (temperature); 50% ± 5% (relative humidity), with water spraying (20 cc every day), after removing from mould until testing date

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All conditionings implied a preparation with the filling of the mould with mortar in two equal layers, with 25 strokes of the tamper each and skimming of excess mortar with a palette knife.

Preparation and conditioning following C2, implying the filling of the mould over a filter paper and placement of another filter paper on top after complete filling of the mould, was thought of in order to absorb excess water in the mortars, with a solution of easy execution.

The daily spraying with water intended an approximation to real conditions, where high night relative humidity prevails throughout the year. Previous tests (Veiga 1997 and 2000) confirmed the adequacy of this method.

Using these methods, hardening of specimens proved sufficient; the only visible negative effect was the wear of specimens on the side previously not in contact with the mould due to water aspersion method.

Other conditions, including a more approximated condition to that proposed by EN 1015-11 are still in progress and will be presented later.

**Figure 1. Specimen submitted to conditioning C3**

## 4 LABORATORY TESTS

A set of laboratory tests was prepared in order to verify the main differences induced by the conditioning situations. Flexural and compression tests were performed so as to conclude upon mechanical properties, water absorption by capillary action informs about the capacity of mortar to resist capillary ascension and is a physical test and, finally, carbonation control (chemical testing) was effectuated in order to assess differences in carbonation speed induced by the different conditionings.

### 4.1 Flexural and compressive strength

Flexural and compressive tests were performed following EN 1015-11, and the results were obtained at the age of 28 days with values as listed below:

**Table 2 – Flexural and compressive strength**

| | ress (N/mm$^2$) | ve Stress (N/mm$^2$) |
|---|---|---|
| 1 | 0.53 | 1.72 |
| 2 | 0.70 | 1.73 |
| 3 | 0.53 | 1.98 |

Figure 2. Flexural test

There are small improvements of mechanical strength for lime mortars with pozzolanic with changes in conditions techniques; the proportioning of a discontinuous humid environment (water sprinkling) and the use of filter paper over and below the mould during the first days after specimen execution seem both to benefit moderately mechanical strength at 28 days.

### 4.2 Water absorption by capillary action

This test was executed following *Cahier 2669-4 CSTB* (CSTB, 1993) in order to facilitate comparison of results with previous tests made by this method. Water absorption coefficient was calculated as the gradient of the graph representing √t in the x-axis and absorbed mass over section in the y-axis between √10 min and √90 min. Results at 28 days were as follows:

**Table 3 – Water absorption by capillary action**

| Conditioning | er absorption coefficient (g/dm$^2$.min$^{1/2}$) |
|---|---|
| C1 | 17.0 |
| C2 | 20.2 |
| C3 | 14.7 |

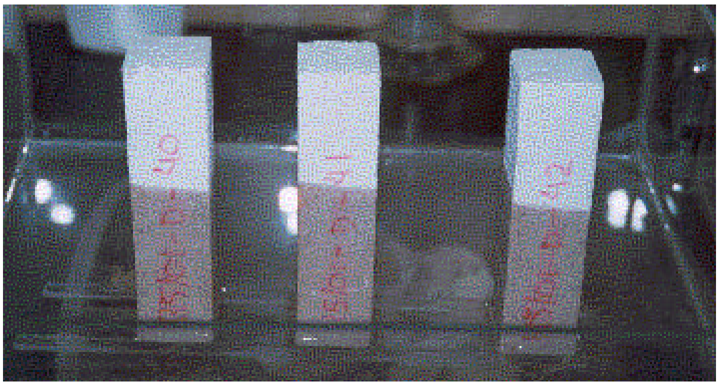

**Figure 3. Testing of water absorption by capillary action**

This test reveals a higher water absorption coefficient for specimens conditioned with the use of filter paper on both sides of the mould and lower values for specimens sprayed with water, relatively to control specimens.

### 4.3 Carbonation control

Carbonation depth was measured using testing procedures described in ICCROM's ARC (Teutonico 1998). The results obtained at 28 days are listed below:

**Table 4 – Carbonation control**

| | ıtion depth (mm) |
|---|---|
| :1 | 9.1 mm |
| :2 | 5.5mm |
| :3 | 11.0mm* |

*- Medium results obtained from previous testing of lime mortars additivated with pozzolans

From these results it might be concluded that there was a decrease in carbonation speed for conditioning involving the use of filter paper on both sides of the mould and an increase in carbonation speed in the case of water spraying during conditioning.

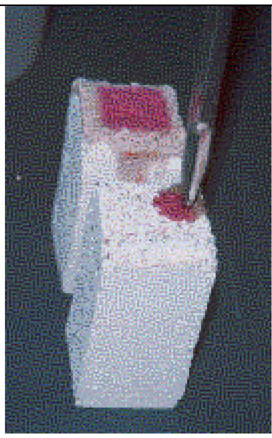

**Figure 4. Carbonation test**

## 5 CONCLUSIONS

Conclusions can be formulated on two levels:

- Preparation and conditioning easiness of execution and output
- Relation between laboratory tests and preparation and conditioning

On the first level, execution of specimens proved easier than EN 1015-11 and effective, as specimens could be easily handled after withdrawal from moulds. However, conditioning C3 caused some wear upon specimens due to the water spraying - this method must be improved.

The tested mechanical properties - flexural and compressive strength – seem to be slightly improved by both the experienced preparation and conditioning changes, but these results must be confirmed with tests at 90 days because they are not completely conclusive.

It may be concluded that the use of filter paper led to a decrease in carbonation speed and an increase in the water absorption coefficient. The decrease in carbonation speed could be explained by a better compacity of the mortar, originated by a diminution of the water that evaporates; in fact, better compacity could reduce the diffusion of $CO_2$; a lower carbonation rate explains a higher water absorption coefficient, because carbonated lime is less water absorbent than non-carbonated.

On the other hand, the spraying of water upon the specimens results in higher carbonation speed and lower water absorption coefficient. In this case, apparently, an increase in the humidity helps to increase the carbonation rate, because that chemical reaction needs some humidity; as a consequence, the mortar becomes less water absorbent.

This small set of tests, included in a larger testing campaign, does not permit great certainties. However, it does indicate that conditioning influences carbonation rate, mechanical resistance and water capillarity and so it is important to continue research so as to conclude about conditions representing reality in a more approximate way.

The development of studies approaching other conditions, similar to EN 1015-11 and a comparison with atmospheric conditions in Summer and in Winter will provide a further and more complete insight into this matter.

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# Programmed Maintenance Procedures Of Staircases Historical Buildings In The Ancient Centre Of Naples

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**Summary: The safeguard and conservation of cultural assets, while representing an important objective for society in an ordinary City context, certainly constitutes to primary task in all those areas subject to seismic risk, such as most of the Campania Region and of Naples metropolitan area in particular.**

**Knowing to building means knowing its history, the natural and City environment around it, the distributive characters, the specific building techniques, and the instruments and materials used for its realization.**

**The planning operation of adaptation of new functions in the stratified texture of the ancient, factory indispensable in order to bring back to new life and to renew the ancient splendor of these buildings, demands particular attention, being bound to the circumstance of having to operated within the limits of the historical, formal, material and technological characters of the building and the context.**

**The knowledge of the traditional building techniques, in particular, becomes fundamental since only the exact determination of to there allows correct actions of recovery and restoration.**

**Another fundamental point is the knowledge of the historical and constructive events that have interested the building, I know as to permit the reconstruction of the evolution that the building organism has had in the course of its life. This also allows for to dating of the single parts and of the entirety, process of fundamental importance since the exact determination of the architectonic formal and functional aspects, supplies the correct indications in the choice of useful functions respectful of the historical and cultural characters of the ancient building; moreover the determination of the materials and the traditional constructive techniques, employed in the construction of the building, or part of it, is fundamental in order to address the choice of the materials and the more appropriate, traditional and innovative techniques, in order to give back the ancient splendor to the building both historical and cultural.**

**In recent years, however, the search has endured to moment of stasis, bound from limits connected to the acquired knowledge; for this reason it has been decided to schedule to research plan that, on the basis of the acquired know-how, induces to the attainment of innovative objective for the conservation and the maintenance of the building, with reference to present and futures the requirements.**

**On based the previous considerations, it has been deemed opportune, therefore, to develop to search that follows to structured system of cultural trails.**

**In particular in this contribution the following matters will be developed:**

- **procedures for the appraisal of the compatibility of actions and innovative techniques with the characteristics of the buildings of ancient manufacture and their context;**
- **procedures for the evaluation of the life expectation of the of recovery actions;**
- **procedures for the evaluation of the sustainability of the materials and techniques used in the recovery action.**

**Keywords: Maintenance, service life, durability, maintainability.**

## 1 INTRODUCTION

In this paper, a procedure for the assessment of service life of historical buildings in the Ancient Center of Naples is presented. This study appears to be very interesting as far as the urban structure of Naples is regarded, since its "Ippodameo" track hasn't changed, during the years (Fig. 1).

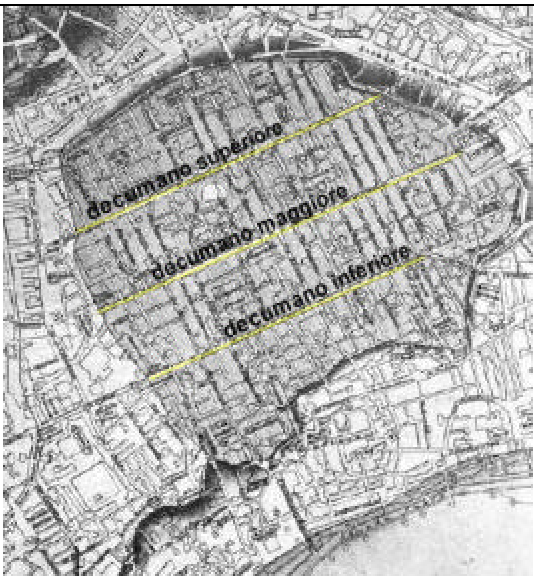



**Figure 1: The Ancient Center of Naples**

**Figure 2: A Neapolitan courtyard**

This study deals with masonry staircases, which have a formal value, not only as a technological element, but also as a component of an articulate and more diffuse system. The present imagine of Ancient Center of Naples is the result of the secular stratification of a structure grown on itself.

Architectural history of the city of Naples developed a strong characterization because of its adaptation to "cardi" and "decumani", which constitute the urban morphology.

The overcoming of the physical limit, represented by this layout, took place through the research of new intuitions, new perceptions, new projects, carried out by the most important Neapolitan architects.

Designers, in order to capture the attention of spectators, devoted themselves to the realisation of plans, which privileged the portal and the system hall-portal-staircase.
In this way, staircase become a most important element inside the building, though being drown back with respect to the street (Fig. 2).

The greater part of Neapolitan staircases presents its facade open on to the court-yard. This choice is due to planning and lighting requirements.

The proposed procedure for the estimate of staircases service life of Ancient Center of Naples, is divided into four phases:

1. Typological analysis and individualisation of more recurrent typologies;
2. Analysis of degradation and evaluation of intervention techniques;
3. Estimate of materials durability and of technologies for interventions;
4. Planning of maintenance interventions in stairs service life.

This study is the result of a combined collaboration between the already cited authors, and so each part is their critical contribution.

Namely, arch. Daniela Marinelli authored the part about Typological analysis and individualisation of more recurrent typologies; prof. Flavia Fascia made the part about The Analysis of degradation and evaluation of intervention techniques; prof. Renato Iovino The Estimate of materials durability and of technologies for interventions and arch. Maria Gabriella Russo The Planning of maintenance interventions in service life of staircases.

## 2 TYPOLOGICAL ANALYSIS AND INDIVIDUALISATION OF MORE RECURRENT TYPOLOGIES***

The choice of a reading methodology that tests the role assumed from the staircase in the Neapolitan environmental system, entails the evaluation of not only static, technological and material attributes, but also of relational attributes.

In order to reach the identification of the staircases dominant types, a typological table has been structured; it is articulated into eight fields, containing information on the typological-formal characters of these episodes of remarkable architectonical value: staircases.

Each table is specific of a particular building to which the staircase belongs.

Beyond to the building location, the eight fields of the table are structured as follows:

DP **-** *Dislocation with respect to the portal.* It is the staircase dislocation in the planimetric system, towards who overpasses the main portal: **1)** frontal; **2)** lateral. The datum provides information on staircase position specifying if the access is from: **a)** the hall; **b)** the courtyard.

*IM* **-** *Architectonic System.* The datum is structured by making staircase typology belong to one of the following classes: **1)** staircase core with wall; **2)** staircase core with pillars; **3)** free staircase core; **4)** closed staircase well; **5)** staircase well with pillars; **6)** free staircase well; **7)** tenail staircase; **8)** fan staircase.

Afterwards, indications on the flights number and the presence and the characterization of staircase raiser are presented.

*AP – Openings.* Through this datum, the relations of staircase body with the surrounding spaces are deduced, inner or external spaces to the building. Firstly, these surrounding spaces are specified: **a)** the road; **b)** the hall; **c)** the inner courtyard; **d)** external spaces. Secondly the composition of the prospects of the staircase emerges through the number of openings: **1)** opened on a side; **2)** opened on two sides; **3)** opened on three sides; **4)** closed.

*Sv – Development.* A typical character of the various tipologies is determined also from the configuration in plant **1)** rectilinear or **2)** curvilinear of each parts of the staircase: **a)** flight, **b)** intermediate foot-pace; **c)** storey foot-pace.

*PP* **-** *Main Elevation.* It indicates the formal aspect of the main elevation specifying the composition: **1)** one arc, **2)** multiples arches **3)** platband. This datum is specific of each the parts of the stair body: **a)** flight, **b)** intermediate foot-pace; **c)** storey foot-pace.

*Rv – Covering.* The covered elements of the body staircase are indicated: **a)** basement; **b)** inner vestment; **c)** step; **d)** under-step; **e)** foot-pace. For each one the type of material used is specified among: **1)** piperno; **2)** pietrarsa; **3)** marble; **4)** majolica; **5)** other.

*Fn – Finish.* The parts of the staircase body with finishing works are indicated: **a)** the base; **b)** the external vestment; **c)** the inner vestment. The finishing works are specified contextually in: **1)** plaster; **2)** stucco; **3)** decoration elements.

*Pr – Protection.* The datum indicates the type of protection adopted for every part: **1)** balustrade, **2)** banisters **3)** hand-rail, specifying the belonging of each member of the body staircase: **a)** flight, **b)** intermediate foot-pace; **c)** storey foot-pace.

The typological tables have been elaborated for 150 building staircases of the Ancient Center of Naples, among the most representative ones.

For example table regarding the Avellino Palace is enclosed. (Tab. 3)

Afterwards, a database has been elaborated that, besides constituting the archives and catalogue of all the staircases examined, has concurred to characterize, with logical statistics, the more recurrent tipologies. Tab.4 shows the database extract regarding the 64 staircases whose tipological characters are dominant.

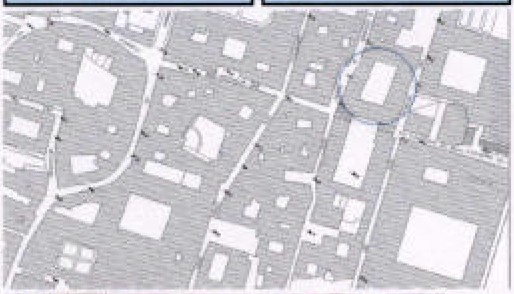
LE SCALE IN MURATURA DEGLI EDIFICI DEL CENTRO ANTICO DI NAPOLI

*Tecniche per il recupero degli elementi non strutturali*

LETTURA E CLASSIFICAZIONE TIPOLOGICA

PALAZZO : Avellino, largo Avellino

SCHEDA N° 87 — SETTORE N° 2



**DISLOCAZIONE RISPETTO AL PORTALE — DP**

| | | | |
|---|---|---|---|
| a androne | 1 frontale | a | |
| b cortile | 2 laterale | b | 2 |

**IMPIANTO ARCHITETTONICO — IM**

| | | | | |
|---|---|---|---|---|
| 1 anima (setti) | 5 pozzo (pilastri) | n° di rampe | 3 | 5 |
| 2 anima (pilastri) | 6 pozzo (libero) | | | |
| 3 anima (libera) | 7 tenaglia | Invito | GRADINO | |
| 4 pozzo (chiuso) | 8 ventaglio | | | |

**APERTURE — AP**

| | | | |
|---|---|---|---|
| a sulla strada | 1 aperta su un lato | a | |
| b sull'androne | 2 aperta su due lati | b | |
| c sul cortile interno | 3 aperta su tre lati | c | 1 |
| d su spazi esterni | | d | |

**SVILUPPO — SV**

| | | | |
|---|---|---|---|
| a rampa | 1 rettilineo | a | 1 |
| b pianerottolo di riposo | 2 curvilineo | b | 1 |
| c pianerottolo di caposcala | | c | 1 |

**PROSPETTO PRINCIPALE — PP**

| | | | |
|---|---|---|---|
| a rampa | 1 arco unico | a | |
| b pianerottolo di riposo | 2 archi multipli (1) | b | |
| c pianerottolo di caposcala | 3 elementi rettilinei | c | 2 |

**RIVESTIMENTO — RV**

| | | | |
|---|---|---|---|
| a basamento | 1 piperno | a | 1 |
| b paramento interno | 2 pietrarsa | b | |
| c gradino | 3 marmo | c | 1 |
| d sottogradino | 4 maioliche | d | |
| e pianerottolo | 5 altro | e | 1 |

**FINITURA — FN**

| | | | |
|---|---|---|---|
| a basamento | 1 intonaco | a | |
| b paramento esterno | 2 stucco | b | 1,2,3 |
| c paramento interno | 3 el. di decoro (1) | c | 1,2 |

**PROTEZIONE — PR**

| | | | |
|---|---|---|---|
| a rampa | 1 balaustra | a | 1,3 |
| b pianerottolo di riposo | 2 ringhiera | b | |
| c pianerottolo di caposc. | 3 corrimano | c | 1 |

**NOTE**

(1) La composizione del prospetto principale propone un arco centrale policentrico di gusto sanfeliciano e due archi laterali a tutto sesto. Tale composizione è stata interrotta al 3° livello da tompagnature che hanno occluso l'arco centrale ed uno laterale. Caratterizzanti la tipologia risultano le chiusure d'ambito con infissi lignei delle aperture del secondo e terzo livello.

**DP b2 / IM 5 / AP c1 / SV a1b1c1 / PP c2 / RV a1c1e1 / FN b1,2,3c1,2 / PR a1,3c1**

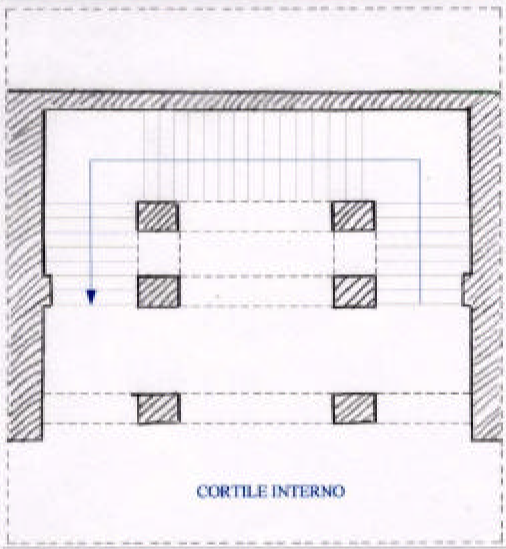



**Table 3: Avellino Palace typological table**

| S | | EDIFICIO | | DP | IM | AP | SV | PP |
|---|---|---|---|---|---|---|---|---|
| 1 | 1 | | via Nilo, 28 | b2 | 1 | c1 d1 | a1 b1 c1 | b1 c1 |
| 2 | 4 | | via S.Biagio, 78 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 3 | 5 | | via S.Biagio, 64 | b1 | 6 | c1 | a1 b1 c1 | c1 |
| 4 | 7 | p. Venezia | via B. Croce | b2 | 5 | c1 | a1 b1 c1 | c2 |
| 5 | 8 | p. De Sangro | p.zza Nilo, 7 | b1 | 1 | c1 | a1 b1 c1 | c1 |
| 6 | 9 | | via Duomo, 45 | b1 | 6 | c1 d1 | a1 b1 c1 | c1 |
| 7 | 11 | | via Duomo, 19 | b1 | 6 | c1 | a1 b1 c1 | c2 |
| 8 | 13 | | via Duomo, 36 | b1 | 6 | c1 | a1 b1 c1 | c1 |
| 9 | 14 | | via Duomo, 50 | b1 | 2 | c1 | a1 b1 c1 | c1 |
| 10 | 16 | | via Duomo, 64 | b1 | 2 | c1 | a1 b1 c1 | c2 |
| 11 | 17 | | via Duomo, 77 | b1 | 6 | c1 | a1 b1 c1 | c2 |
| 12 | 26 | | via Duomo, 276 | b1 | 2 | c1 | a1 b1 c1 | c2 |
| 13 | 27 | | via Duomo, 254 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 14 | 31 | | via Duomo, 214 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 15 | 32 | | via Duomo, 213 | b1 | 6 | c1 | a1 b1 c1 | c1 |
| 16 | 34 | | via Duomo, 202 | b2 | 2 | c1 | a1 b1 c1 | c1 |
| 17 | 35 | | via Duomo, 196 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 18 | 42 | | via S.Geronimo, 29 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 19 | 45 | | via S.G. Pignatelli, 8 | b2 | 4,6 | c1 | a1 b1 c1 | c1,3 |
| 20 | 50 | | via R. De Sangro, 4 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 21 | 51 | | via R. De Sangro, 27 | b2 | 1 | c1 | a1 b1 c1 | b1 c1 |
| 22 | 54 | | via Paladino, 2 | b1 | 1 | c1 | a1 b1 c1 | b1 c1 |
| 23 | 55 | | via Paladino, 5 | b2 | 2 | c1 | a1 b1 c1 | a1 b1 c1 |
| 24 | 56 | | via Paladino, 9 | b1 | 2 | c1 | a1 b1 c1 | a2 b2 c2 |
| 25 | 58 | | via Paladino, 25 | b2 | 2 | c1 | a1 b1 c1 | c1 |
| 26 | 59 | | via S. Marcellino, 5 | b1 | 6 | c1 | a1 b1 c1 | c2 |
| 27 | 60 | p. Carafa della Sp. | via B. Croce, 45 | b2 | 2 | c1 | a1 b1 c1 | a1 c1 |
| 28 | 73 | p. Bonifacio | via Portanova, 15 scala B | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 29 | 74 | p. Petrucci | p. S. Domenico | b1 | 1 | c1 | a1 b1 c1 | c2 |
| 30 | 75 | p. Tufarelli | via B. Croce | b1 | 5 | c1 | a1 b1 c1 | c2 |
| 31 | 76 | | via Paladino, 6 | b1 | 6 | c1 | a1 b1 c1 | c1 |
| 32 | 78 | | via Purgatorio ad A., 7 | b1 | 5 | a1c1 | a1 b1 c1 | c2 |
| 33 | 79 | p. Avellino | l.go Avellino | b2 | 5 | c1 | a1 b1 c1 | c2 |
| 34 | 81 | | via Tribunali, 197 | b1 | 5 | c1 | a1 b1 c1 | c1 |
| 35 | 83 | | via Tribunali, 169 | b1 | 5 | c1 | a1 b1 c1 | c1 |
| 36 | 84 | | via SS. Apostoli, 3 | b1 | 5 | c1 d1 | a1 b1 c1 | c1,2,3 |
| 37 | 86 | p. dei Loffredi | vico Loffredo, 7 | b2 | 5 | c1 | a1 b1 c1 | c1 |
| 38 | 87 | Imperatore di Cost. | via Tribunali, 339 | b2 | 1 | b1 c1 | a1 b1 c1 | c1 |
| 39 | 88 | | p.zza S.Gaetano, 322 | b1 | 1 | c1 | a1 b1 c1 | b1 c1 |
| 40 | 91 | | via Duomo, 179 | b1 | 6 | c1 | a1 b1 c1 | c1 |
| 41 | 93 | | via Duomo, 167 | b2 | 6 | c1 | a1 b1 c1 | c1 |
| 42 | 96 | p. Casamassima | via Banchi Nuovi, 8 | b2 | 1 | c1 d1 | a1 b1 c1 | c2,3 |
| 43 | 97 | p. Bonifacio | l.go Regina Coeli | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 44 | 98 | | via S.G.Pignatelli, 29 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 45 | 99 | | via Nilo, 22 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 46 | 101 | | via Nilo, 20 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 47 | 103 | | via S. Biagio, 25 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 48 | 104 | | via S. Biagio, 14 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 49 | 105 | | via Tribunali, 181 | b1 | 2 | c1 | a1 b1 c1 | c2 |
| 50 | 106 | | p. Sedil Capuano 243 | b1 | 1 | c1 | a1 b1 c1 | c1 |
| 51 | 108 | | v. dei Maiorani 17 | b2 | 1 | c1 d1 | a1 b1 c1 | c1 |
| 52 | 109 | | v. dei Maiorani 9 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 53 | 112 | | via S. Nicola a Nilo 13 | b2 | 6 | c1 | a1 b1 c1 | c1 |
| 54 | 117 | | via SS. Filippo e Giac., 29 | b2 | 6 | c1 | a1 b1 c1 | b1 c1 |
| 55 | 120 | | via S.Greg. Armeno 41A | b2 | 1 | c1 | a1 b1 c1 | b1c1 |
| 56 | 127 | | vico Zuroli, 42 | b1 | 6 | c1 | a1 b1 c1 | c1,3 |
| 57 | 128 | | vico Gerolomini, 11 | b1 | 5 | c1 | a1 b1 c1 | c1,3 |
| 58 | 129 | | vico Vertecoeli, 9 | b2 | 1 | c1 | a1 b1 c1 | c1 |
| 59 | 130 | | vico Sedil Capuano, 21 | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 60 | 134 | | vico Sedil Capuano, 6 | b2 | 6 | c1 | a1 b1 c1 | c2 |
| 61 | 135 | | vico Sedil Capuano, 4 | b1 | 1 | c1 | a1 b1 c1 | c2 |
| 62 | 138 | | via Tribunali 276 | b1 | 1 | c1 | a1 b1 c1 | c1 |
| 63 | 139 | p. Mormando | via S Gregorio Armeno | b2 | 1 | c1 | a1 b1 c1 | c2 |
| 64 | 140 | | via S.Arcangelo a Baiano, 44 | b1 | 1 | c1 | a1 b1 c1 | c2 |

**Table 4: Database extract**

## 3 ANALYSIS OF DEGRADATION AND EVALUATION OF INTERVENTION TECHNIQUES*

The second phase of the research is focused to the types of dominant staircases emerged in the first phase. In order to characterize degradation pathologies, the authors have designed "technological tables" that examine the material and tecnological characteristics of the single elements, defining the staircase. For each element, the table provides information about the degradation pathologies and the evaluation of their spreading state.

The study fields are structured as follows: firstly, the indication of the technical element analyzed and the alphanumeric code, referring to the staircase it belongs is reported. Successively, there are fields regarding information on: the element description; the position inside the staircase body; dimensional, geometric and constructive data. In a second part, all the possible degradation phenomena that are commonly found in the Neapolitan context are listed. For each phenomenon, the interested layers (element, support, anchorage, covering and finish) and its spreading (25%, 50%, 100%) are indicated.

The order is alphabetical, indipendently from classification or link criteria of the described phenomena. Every degradation phenomenon is illustrated with a significant photographic documentation, and with a definition of the specific bibliography. The listed expressions, even if typical of the stone materials alterations, are used for the other material present in a staircase body, like wood and metal.

The macrocospic alterations of the single elements have been defined only with a careful visual observation of building components.

Given the complexity of the studies, a proper field has been previewed where to annotate other possible information.

For example, the table regarding the balustrade of Palazzo Avellino staircase are enclosed.(Tab.5)

Subsequently, a database has been elaborated that has concurred to characterize, with logical statistics, more recurrent degradation pathologies for each significant element. Tab.5 shows the database extract, regarding the dominant pathologies.

For each pathology, possible intervention techniques have been enlisted, among which only the designer will be able to choose the one compatible with the particular case, object of recovery.

For example the technical table on marble covering pathologies is enclosed.(Tab. 7)

## 4 ESTIMATE OF MATERIALS DURABILITY AND OF TECHNOLOGIES FOR INTERVENTIONS**

From the time scientific conservative techniques and quality control have been taken into consideration, the attention of the scientific research has moved its interest field towards technological requirement, service levels, technical certificates, tests modalities and rules.

In the past, the "quality" of a conservation or restoration project was judged from the so-called "art rules", that contained oral and written instructions, which the intervention had to comply. They were not formalized rules, derived from the experience and handed in the time above all by the labor. These regarded the use of materials and the intervention techniques, such as, for instance, the procedures of cleaning, the consolidation, the plastering and the protection of the art works.

The introduction of new materials and execution techniques, not reportable to the traditional ones, has rendered the concept of "art rule" insufficient, and has rendered the introduction of a related normative technique necessary.

Since the 80s, the Restoration Institute Center in Rome has started the distribution of Normal Recommendations on technical norms regarding the historical and artistic heritage. From that moment a clear instrument has been used everytime there has been a restoration problem.

Afterwards, the third phase of the procedure features a program of experimental tests aimed at assessing the quality and, therefore, the durability of the building element after the possible intervention techniques. Evidently, this evaluation will be conditioned by the compatibility between the intervention and materical and technological characteristics, specific of Neapolitan staircases.

The tests, to be performed in the laboratory of the Department of Building Engineering of the University of Naples Federico II, will be substantially scheduled as follows:

- radiation tests;
- tests of temperature (only cold, only warm, warm-cold);
- climatic tests (constant climate, variable climate);
- cycles of salt spray followed from drying phases with low relative humidity;
- continuous salt spray (conventional salt fog);
- alternated salt spray
- corrosion tests.

These tests are carried out using some specific machines: climatic rooms model Challenge 250 (Angelantoni industries), dry corrosion test cabinet model DCTC 600.

## 5 PLANNING OF MAINTENANCE INTERVENTION OF STAIRCASES IN THE ANCIENT CENTRE OF NAPLES

Planning of maintenance activities is the expression of the last phase of the study executed on the staircase, which is the protagonist, with its hardy forms, of facades and courtyards of the greatest Neapolitan palaces. In particular, this study follows the individualisation of most recurrent types, the analysis of the degradation, the detection of the intervention techniques and the definition of the materials durability.

This paper, as a result of a combined collaboration between the cited authors, refers to the study of the role of programmed maintenance with respect to the staircase, and the reference to a possible maintenance planning, regarding the staircases service life.

The normative table regarding the programmed maintenance offers many hints for a reflection; however, it is important to cite, in particular, the Merloni Law, 109/94, and its modifications and integrations, in which: *maintenance handbook*, *user's manual* and *maintenance program* represent building maintenance instruments. In particular, they constitute operative instruments for *maintenance plan.*

Namely, this maintenance plan aims at realising a correct use of the heritage, with a rational and simplified planning of building activities.

From an operative point of view, it is important to individualise a methodology for the appraisal of the state of conservation of buildings, or of the single technological units; this is a fundamental phase for the passage to the next one which is the choice of the intervention techniques.

The cited methodology requires, firstly, the individualisation of the instruments necessary for measuring the degradation and the analysis of conditions of building components; after, it requires the realisation of a model regarding building maintenance.

Before reporting about maintenance interventions planning, regarding staircases, it is important to specify that there are many imperfections which can disturb the conservation state of building components, during its service life, and naturally many are the causes that determine its manifesting.

It is important to specify that the execution of the inspections foreseen by maintenance manual is a fundamental condition for the successive applicability of activities defined by the maintenance program.

Now, backing to Neapolitan staircases, it emerged that between the more recurrent types of staircases in the Ancient Center of Naples there are: “core staircase” and “well staircase” while the most used materials are: “tuff”, as a construction stone, “piperno” and marble. These materials are utilised in external bases, balusters, treads, decorative elements.

Qualities of resistance and long duration are important characteristics of stone materials. Natural ageing, usury and other pathology do not compromise the material use. The recovering intervention is due to a lot of factors, such as: subjectivity of the designer, specific requirements of ambients, representativity of staircase.

The most important degradation pathologies that attack stone materials and, as a consequence, staircases, can be divided into three categories: smaller, serious, graves[1].

The first ones are connected to superficial aspect of elements; the second ones are connected to the superficial aspect and to the functional aspect of elements; the third ones, which are the most detrimental, seriously compromise elements services.

The smaller “defects” are: superficial erosions, spots, superficial deposit with, as a consequence, superficial soiling, biological patina (fungus, lichens, musks), efflorescences, holes, cracks, edges chipping, superficial splits.

The serious “defects” are: abrasions, big superficial damages, bending and loss of planarity, as a consequence, of superficial layers.

Finally, the graves “defects” are: through cracks and holes which attack the whole facing thickness, elements detachment, humidity (fig. 6). This last one, like an ascent humidity, is due to the presence of cavities, in the subsoil of Naples. This type of humidity, moreover, is show itself in the first staircase flight. Next to these defects there are others, due to the usury of treads facing , which attack the first staircase flight. The most important effect of this type of usury, due to the use of the staircase, is a sinking of the central part of treads and a loss of planarity, as a consequence, of superficial layers.

Now, it is important to specify, in this last case, marble stairs from “piperno” stairs. In a restoration intervention, it is possible, in fact, to replace the whole or only a part of a marble treads because the marble is a easy to find material; in “piperno” staircases, its substitution is unimaginable because this type of stone is very difficult to find.

After individualising defects of technological element and after defining its intensity, on a base of diffusion thresholds of the same element, it is possible to assess the possibility of executing the maintenance intervention.

Particularly, this intervention could be a corrective type or a substitutive type.

In this case, the temporal schedule of maintenance interventions is very important. We have *programmed maintenance*, but also we have preventive forms of maintenance. Particularly, *under-condition maintenance*, which presupposes periodic controls, aimed at the evaluation of degradation; for example, analysis of superficial layers of elements and intervention frequency. *Corrective maintenance*, which presupposes the execution of repairing interventions that follow the diagnosis of defect causes. It is desirable the combined adoption of these maintenance strategies in order to provide, in the best way, a political maintenance and so a tutelage of heritage, during the years.

| ANALISI DEL DEGRADO DEI MATERIALI | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | FENOMENO | STRATI INTERESSATI | | | | | DIFFUSIONE | | |
| | | | | | | | Bassa | Media | Alta |
| | Alterazione cromatica | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Alterazione cromatica da dilavamento | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Alveolizzazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Deposito superficiale o crosta | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Distacco latente | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Efflorescenza | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Erosione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Erosione meccanica | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Esfoliazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Fessurazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Macchia | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Mancanza | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Patina biologica | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Polverizzazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| * | Presenza di vegetazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Rigonfiamento | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Umidità da infiltrazione | elemento | supporto | ancoraggio | rivestimento | finitura | | | |
| | Umidità da risalita | elemento | supporto | ancoraggio | rivestimento | finitura | | | |

**Table 5: Database extract of staircases degradation pathologies**

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# LE SCALE IN MURATURA DEGLI EDIFICI DEL CENTRO ANTICO DI NAPOLI

*Tecniche per il recupero degli elementi non strutturali*

## SCHEDA TECNOLOGICA - scala p. Avellino

| ELEMENTO TECNICO | BALAUSTRA |
|---|---|

***Codice di riferimento:*** **DP b2/IM 5/AP c1/SV a1b1c1/PP c2/ RV a1c1e1/FN b1,2,3c1,2/PR a1,3c1**

***Descrizione:*** la balaustra presenta un basamento in piperno, una cimasa ancora in piperno e colonnine sagomate in marmo.

***Posizione:*** rampa interna intermedia

***Dati dimensionali:*** altezza basamento 20 cm, colonnina 68 cm, cimasa 20 cm, sezione 20 cm.

***Dati geometrici:*** lastre rettangolari sagomate a guscio, colonnine con raggio crescente verso il basso.

***Dati costruttivi:*** lastre in materiale lapideo connesse mediante elementi metallici e giunti di malta.

## ANALISI DEL DEGRADO DEI MATERIALI

| FENOMENO | STRATI INTERESSATI | | | | | DIFFUSIONE | | |
|---|---|---|---|---|---|---|---|---|
| Alterazione cromatica | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Alterazione cromatica da dilavamento | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Alveolizzazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Deposito superficiale o crosta | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Disgregazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Distacco latente | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Efflorescenza | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Erosione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Erosione meccanica | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Esfoliazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Fessurazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Macchia | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Mancanza | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Patina biologica | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Pitting | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Polverizzazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Presenza di vegetazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Rigonfiamento | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Umidità da infiltrazione | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |
| Umidità da risalita | supporto | elemento | ancoraggio | rivestimento | finitura | 25% | 50% | 100% |

**NOTE:**

**Table 6: Technological table of Avellino Palace balustrade**

Causes

In the past it was a common idea that the most important causes of degradation were due to physical aspects (thermal excursion, ices, action of the wind, etc).

Today, instead, recent studies have enlighted the influence of chemical (atmospheric pollution) and biological aspects to the degradation of marble.

Actions which can cause degradation can be divided into these principal groups:

physical actions (thermal, of humidity, mechanics, of the wind);

chemical actions ("silicati" alterations, minerals);

biological actions (macro and micro-organism).

Physical actions

- Water actions

It is possible to find inside the stone a hygroscopic water amount, that can vary with the hygrometrical degree, and percolation water or capillary. The soaked water can cause damages to marble: it favours chemical and biological processes, indirectly, it provokes alterations due to the relations with other materials which it is in contact with (plaster, mortar, metal).In all cases, the modifications made by the water weaken the structure and alter cohesion.

- Particle sediments present in the atmosphere

In the atmosphere, particularly in polluted atmosphere, in which pollution is due to vehicles traffic, suspensions of inorganic and organic powder, hydrocarbon molecules unburnt ometimes present and can deposit on marble, causing degradation of this material.

- Thermal expansion

The temperature variations provoke expansion and contraction of the material; large thermal excursions can cause expansions-contractions that may determine local disintegrations. The disintegrations are more serious when different minerals are present in the marble specimen having different thermal expansion coefficient.

Chemical actions

The chemical degradation is attributed mainly to the atmosphere, and in particular to oxygen, carbon dioxide and the water. These substances, in combined action are particularly aggressive towards every type of marble. Most of the stone materials used in construction are composed of silicates or carbonates, of which we can have the following alterations:

- *Alteration of the silicates.*

They take place on cliffs composed mainly of $SiO_2$ (granites, basalts, porfidi and sandstones).

- *Alterations of carbonates.*

They take place on cliffs composed of calcium carbonate, and on those in which the carbonates are present in variable percentages, which are susceptible to acid attack. The effect of the chemical aggression is visible in the superficial corrosions of the material, and in the white or black crust.

- *Alterations and spots due to iron minerals.*

In white marbles, above all, spots of filterings can manifest, due to pyrite oxidation or siderite solutions, that can be present, even in small part, in inclusions or venatures. The more obvious effect is the alteration of color (from very light, bright yellow to reddish brown tones, which can affect marbles, on the outside as well as on the inside of buildings.

Biological actions

The biological aggressions are caused by macro-organisms as well as micro-organisms, of vegetable and animal origin. Among these we can remember the damage caused by the roots that penetrate the fissures in cliffs; the damage caused by birds that deposit excrements corrosive to limestone cliffs; the damage caused by bacteria, (like in the case of the "diseases of plates") by mosses, by licheni, by alghe, etc.

**Table 7: Pathologies of coverings in marble**

# Monitoring For Preservation Of Antarctica's Historic Buildings

WD Ganther[1] JD Hughes[2] V Daniel[3] & IS Cole[1]
[1]CSIRO Building, Construction and Engineering Highett Victoria
[2]National Gallery of Australia, Canberra [3]Australian Museum, Sydney

**Summary: It is well known that to produce a durable structure, the conditions affecting the structure in its environment must be known before specifying the component materials and design of that structure. Also, when preserving a structure that is already built, it is important that the conditions producing deterioration of the structure be investigated in addition to knowing the current condition of the component materials.**

**There is a common belief that the cold dry climate of Antarctica will prevent problems such as biodeterioration and corrosion, but the expectation of 'perfect preservation' has been shown to be a myth. To preserve historic buildings such as the huts of the early Antarctic explorers, it is essential to understand the processes by which deterioration occurs and to accurately measure the rate at which it occurs. This information in general will be of great benefit in devising long-term options for site conservation.**

**This paper provides a case study of monitoring and materials research undertaken at Mawson's Hut, which was built in 1912 at Cape Denison in the Australian Antarctic Territory. The research focussed on the acquisition of temperature and relative humidity conditions, corrosion, salt deposition and erosion of timbers.**

**Keywords: Antarctica, monitoring, historic buildings, Mawson's Huts.**

## 1 INTRODUCTION

This paper examines the monitoring of environmental conditions at Mawson's Hut at Cape Denison to illustrate the importance of understanding the deterioration processes affecting the building structures and artefacts by wind, moisture, salts, ice abrasion and solar radiation.

This building was hurriedly constructed during January 1912 at Cape Denison (67°S 142°E) in the remote Eastern Sector of the Australian Antarctic Territory. Being associated with internationally significant early scientific expedition in Antarctica it is protected under the Antarctic Treaty. The site, which has four buildings and numerous scattered artefacts, is particularly important to Australians because of the heroism of its leader, the scientist Professor Sir Douglas Mawson.

Cape Denison suffers extreme weather: Mawson named it the 'Home of the Blizzard' because of the severe katabatic winds, which make it one of the windiest places on earth. Temperatures are below –20°C in winter, and occasionally above 0°C during summer, leading to melt water formation. Winds carrying ice particles erode the wooden cladding and some structural damage has occurred.

Various proposals have been made since the 1970s for conserving Mawson's Hut. These include covering the building with a transparent dome, over-cladding of the original exterior timbers, dismantling for relocation to Australia, and various options for removal of the ice that has been blown inside the building over the years. Removal of ice is perhaps the most frequently proposed conservation option and this has been proposed by the Australian Associated Press Foundation (AAPF) to allow tourist access (AAPF 1999).

The monitoring system installed in the building in 1999 is briefly described, as well as the difficulties and limitations of operations in this severe climate. The data collected should be used to aid decisions for future management of the physical structure of the buildings and their contents. Ongoing monitoring should also be used to ensure any works carried out are effective and provide information to reassess the management plan in the future.

The most important reasons for carrying out monitoring are:

- to obtain reliable information on temperature and relative humidity in a hostile environment (note there is no detailed information for this site);
- to compare the variability of temperature and humidity conditions in ice-filled and excavated parts of the hut; and
- to use the data to develop a climate model for the building to determine whether and how ice removal may affect the temperature and humidity conditions in the building.

### 1.1 Monitoring methods and equipment

Due to the remote location of the site and the low temperatures, the operating life of the battery, which powers the data-logging system, was a major limitation. It was therefore necessary to balance the need for data from a sufficient number of locations throughout the building with the need for a robust operating system. Given that the battery would be expected to last just over one year, it was anticipated that data collected onto a data card could be downloaded when a ship was expected to be able to reach the site in summer. The site is only accessible by sea, which takes approximately one week from Hobart. The system was specially adapted for operation in polar conditions with seals to protect against ingress by fine ice particles carried by the wind. Monitoring was only carried out in the Main Hut, which is the largest and arguably most significant building at the site.

A Campbell Scientific CR10X monitoring system was installed with eight combined Vaisala temperature/RH sensors and 10 surface temperature thermocouple sensors. This enabled a range of locations from floor to apex and from outer walls to the core of the building to be monitored, thus facilitating an understanding of the differing microclimatic conditions in the hut. Figure 1 shows the location of the sensors.

During 1999, low-alloy copper steel coupons were exposed on top of a building, Gadget Hut, near Mawson's Hut. Low-alloy copper steel (also referred to as copper-bearing steel) is one of a number of standard metals used by CSIRO for atmospheric corrosion studies (King *et al.* 2001a) and has been used in other Antarctic locations (Hughes *et al*. 1996).

### 1.2 Accuracy limits

At one location (bunk in living section, sensor location 6), a Vaisala combined T/RH sensor was placed adjacent to the thermocouple to ensure that any gross anomalies in trends could be identified. The accuracy of the humidity sensors is ±3% at best and ±0.5° for the temperature sensors.

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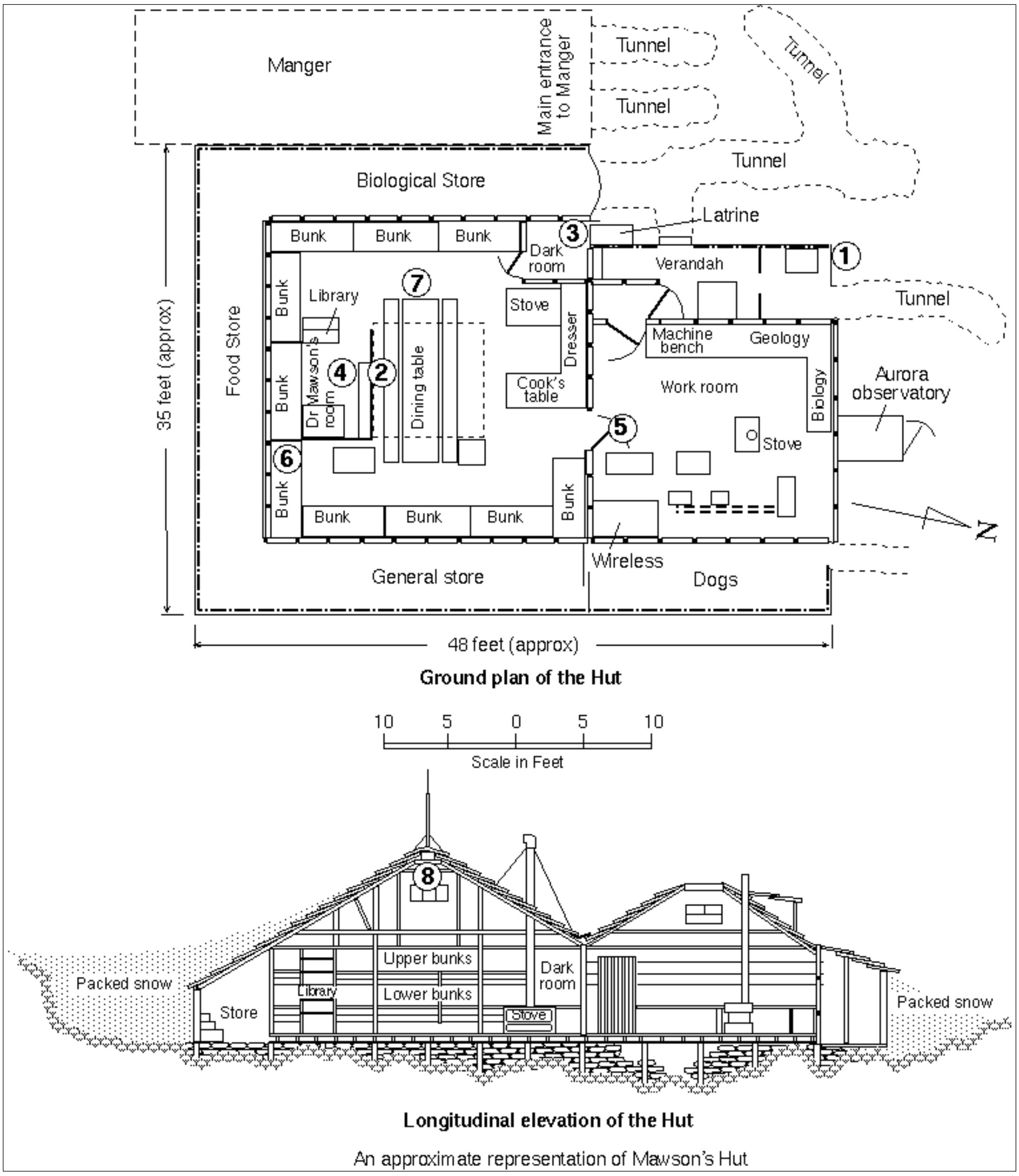


**Figure 1. Location of sensors in Mawson's Hut during 1999 (numbers refer to Table 1)**

## 2 DISCUSSION

Due to the large volume of data that has been collected over the period January 1999 to January 2000 only a summary of the data is presented here. Table 1 is a summary of the microclimate measurements made in Mawson's Hut during 1999. Also, in the table are calculations based on the temperature and relative humidity measurements. Time of wetness (TOW) has been calculated based on ISO 9223 (1992) and by the method described by King *et al.* (2001b). The table also contains calculated surface equilibrium moisture content (SEMC) (Bramhall 1979) of a standard piece of timber if it were in the position of the sensor. Table 2 gives summary data from the thermocouples, which were mainly surface temperature measurements.

Table 3 shows the results from the steel corrosivity samples exposed during 1999. The average atmospheric corrosion rate was 12.2 μm/year, which is of a similar rate to suburban Melbourne or Sydney.

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**Table 1. Summary statistics for monitoring at Mawson's Hut**

| | *1 Exterior* | *2 Living section* | *3 Shelf in Hurley's dark room* | *4 Mawson's bookshelf* | *5 Work-room* | *6 Bunk in living section* | *7 On shelf in living area* | *8 Near Apex in Roof* |
|---|---|---|---|---|---|---|---|---|
| Maximum RH (%) | 103 | 100 | 96 | 99 | 101 | 96 | 100 | 101 |
| Average RH (%) | 85 | 89 | 85 | 89 | 91 | 88 | 90 | 87 |
| Minimum RH (%) | 23 | 56 | 76 | 80 | 84 | 81 | 60 | 56 |
| Maximum temperature (°C) | 8.1 | –0.3 | –0.4 | –0.4 | –0.3 | –0.5 | –0.4 | 3.3 |
| Average temperature (°C) | –14.0 | –14.4 | –14.9 | –14.8 | –13.7 | –14.9 | –14.4 | –13.5 |
| Minimum temperature (°C) | –32.2 | –23.1 | –24.3 | –22.8 | –21.3 | –22.2 | –22.8 | –24.4 |
| ISO TOW (% time) | 2.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 |
| TOW RH >50; T >–10 (% time)* | 31.8 | 28.0 | 26.7 | 27.3 | 29.2 | 27.9 | 28.0 | 31.5 |
| Maximum SEMC (%) | 34.1 | 32.3 | 26.7 | 30.2 | 33.6 | 26.7 | 33.5 | 33.4 |
| Average SEMC (%) | 21.8 | 23.4 | 20.8 | 22.8 | 24.7 | 22.2 | 23.5 | 22.3 |
| Minimum SEMC (%) | 5.9 | 19.4 | 16.8 | 19.1 | 20.6 | 19.1 | 12.7 | 11.8 |
| Time above 16% EMC (%) | 88.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 95.2 |
| Time above 20% EMC (%) | 71.2 | 94.4 | 69.3 | 92.7 | 100.0 | 93.3 | 97.9 | 81.9 |
| Maximum AH ($g/m^3$) | 5.2 | 4.4 | 4.0 | 4.5 | 4.6 | 4.2 | 4.6 | 5.7 |
| Average AH ($g/m^3$) | 1.7 | 1.7 | 1.5 | 1.6 | 1.8 | 1.6 | 1.7 | 1.8 |
| Minimum AH ($g/m^3$) | 0.3 | 0.7 | 0.6 | 0.7 | 0.8 | 0.7 | 0.7 | 0.6 |

* From King *et al.* (2001b).

**Table 2. Summary of surface temperature measurements at Mawson's Hut**

| *Temp. (°C)* | *Acetylene equipment store (TC1)* | *Wall in Mawson's room (TC2)* | *Wooden shelf (TC3)* | *1 m of TC3 (TC4)* | *Hurley's dark room (TC5)* | *Apex of living section (TC6)* | *Between timbers, Mawson's cubicle near bookshelf sensor (TC7)* | *In roof cavity at main entrance (TC8)* | *In roof near TC8 (TC9)* | *2/3 m from floor (TC10)* |
|---|---|---|---|---|---|---|---|---|---|---|
| Max. | –0.4 | –0.5 | –0.4 | –0.4 | –0.4 | 3.2 | –0.2 | –0.1 | 0.3 | –0.4 |
| Ave. | –14.6 | –14.8 | –14.7 | –14.7 | –14.9 | –13.8 | –14.8 | –14.4 | –13.9 | –14.6 |
| Min. | –22.9 | –21.4 | –22.8 | –22.9 | –23.4 | –23.6 | –22.9 | –24.6 | –23.4 | –23.0 |

**Table 3. Low-alloy copper steel corrosion rates at Gadget Hut exposed from 10/3/1999 to 26/12/1999**

| | *Corrosion rate (µm/year)* |
|---|---|
| Coupon 1 | 12.8 |
| Coupon 2 | 11.5 |
| Coupon 3 | 12.4 |
| Average | 12.2 |

The summary information presented in Table 1 shows that the temperature in the hut never gets above 0°C, except in the apex of the Hut, at any time in the year, and that in the majority of locations the relative humidity never falls below 80%. Using the temperature and relative humidity criteria of ISO 9223 (T >0°C and RH >80%), then TOW is zero hours in the hut, except in the apex and outside where it is 2%. This implies that there should be no corrosion inside the hut, which is inconsistent with observations of significant corrosion affecting metal artefacts and the measured corrosion rate of 12 µm/year. This corrosion rate is comparable to corrosion rates found in urban Melbourne (King *et al*. 1981) where TOW is close to 50%.

King *et al.* (2001b) showed that free water capable of corrosion is present at times corresponding to T >–10°C and RH >50%. Using this alternative TOW criteria, the TOW becomes 30% in all locations, which is sufficient for significant corrosion to occur. Thus, the criterion proposed by King *et al*. is more consistent with observation.

The high RH measured inside the building implies that the timber of which the hut is built will also have a high moisture content. The SEMC of an 'ideal' piece of timber can be calculated from the temperature and RH. Decay in timber is said to be possible above 20% equilibrium moisture content (EMC) (Singh 1994) and the SEMC measured in the hut is above 16% for 100% of the time, and above 20% more than 80% of the time. If decay is possible at these temperatures, there is more than enough water available for it to proceed. The studies of Greenfield (1981; 1982) on historic materials at the Scott and Shackleton Huts at an even more southerly location suggest this is indeed possible.

The maximum SEMC is near to or above 30%. This is the value when free water can be found in the timber in the voids and not bound to cells. When water is present in the timber bound to cells, it will not freeze at 0°C but at significantly lower temperatures, possibly as low as –40°C. Free water may freeze at 0°C and can cause disruption to the structure of the timber. Defibring has been observed at many Antarctic historic sites by Hughes, although at some sites the loose fibres are blown away by the wind. Research in Australia by Wilkins and Simpson (1988) suggests that defibring is primarily dur to high salt deposition and is accelerated by high temperatures and humidities, so its existence in Antarctica is curious. It may be that similar crystal growth formation processes in both freezing and salt deposition cycles may be responsible for the observed defibring, as high salt deposition rates are evident at many historic sites, most of which are very close to the sea.

While all the sensors exhibit similar ranges of temperature and RH, except for the external sensor, a limitation of the data is that it does not show whether deleterious conditions such as condensation cycles may be occurring. To examine the likelihood of such problems, temporal plots of the sensor data were produced graphing all the data from a particular sensor in one plot. The x-axis is the time of the year and the y-axis is the time of the day, so each line running vertically is one particular day. This graphing technique enables cycles that function on a daily, seasonal or yearly basis to be easily seen. Figure 2 shows the temporal plots for temperature and humidity for sensors 1, 2, 5 and 8, while Fig. 3 has the temporal plots for the thermocouples. Not all the sensor plots are presented in this paper.

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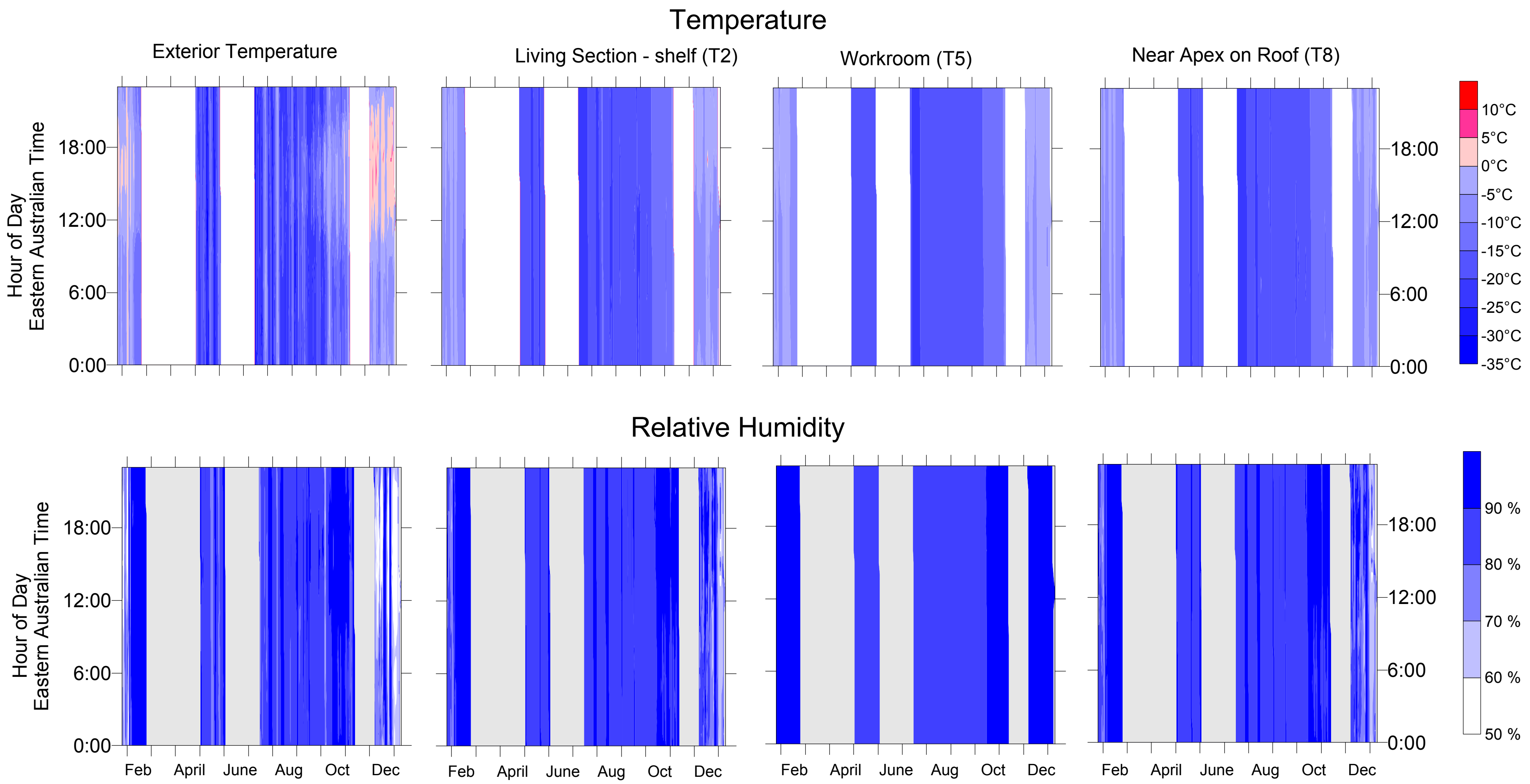


**Figure 2. Temperature and humidity temporal plots for sensors 1, 2, 5 and 8 at Mawson's Hut, Antarctica**

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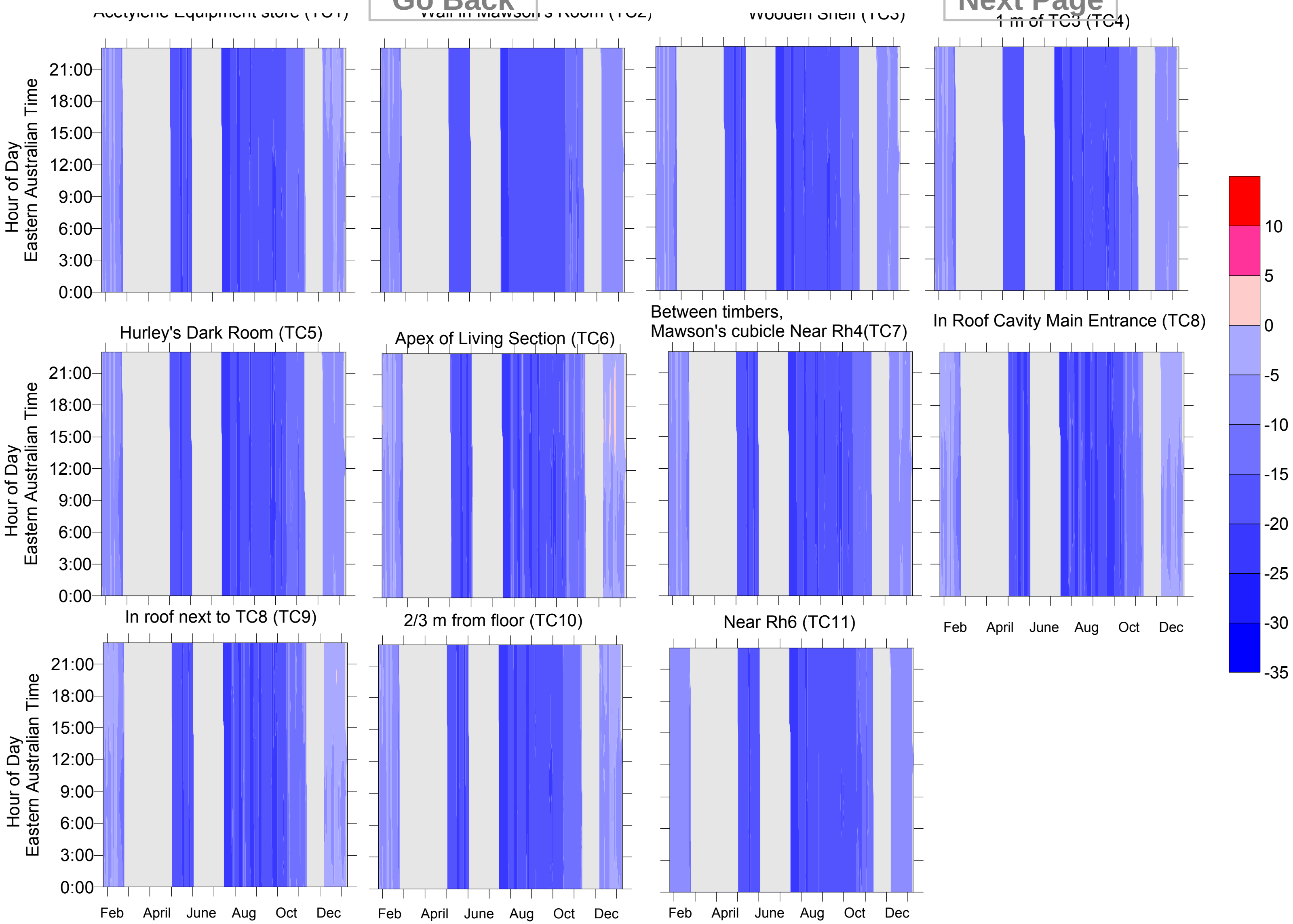


**Figure 3. Temperature temporal plots for thermocouple sensors at Mawson's Hut, Antarctica**

While some data is missing during March, April, June, part of July and part of November, the data shows that strong daily cycles affect the exterior of the hut during the summer (December to February). The existing data is representative of all major climatic seasons at Mawson's Hut and presents a clear picture of the conditions experienced. Thus, the data can be used for interpretation as expressed by Daniel *et al.* (2000). The 'dampening' of these temperature and RH fluctuations by the building are also evident, with the temperature variations being less than 5°C for weeks and months at a time, especially during the winter months. During October, the outside temperatures start to warm up and the corresponding RH shows a very high level during this period. When the temperature is at its highest during December and January, the RH is more variable.

Figures 4–9 are seasonal decompositions of different periods of the data, i.e. each graph shows what the average day would look like for the period shown in the graph. These graphs show the daily cycles that are occurring at the different locations. As the data is averaged, only true daily trends are shown and any events that do not occur each day are averaged out. This also means that the extremes of the cycles are also removed and we do not see the maximums and minimums shown in the summary tables. The weather at Mawson's Hut is extremely variable and this analysis of the data needs to be used in combination with other analysis of the data to determine the complete picture.

Strong daily cycles can be seen in the two summer periods of January to February 1999 and December 1999 to January 2000. However, in the winter period there is little change over the average day, as would be expected without the influence of the sun.

While there are some slight differences between the January to February 1999 and December 1999 to January 2000 periods, the daily cycles are similar enough to be discussed together. For the summer period daily cycles are present, although the daily variation is less then in temperate climates. Only the January to February 1999 graphs are presented in this paper.

The exterior shows the common daily cycle of lower temperatures at 'night' even though the sun does not actually set, and higher temperatures when the sun is at its highest (Fig. 4). The opposite is true for RH, which is low during the day, as warm air can hold more moisture, and high at night (Fig. 6).

Inside the hut the same temperature cycles occur, but are slightly lagged due to the time taken to heat up the structure. The maximum temperatures inside the hut are at 19:00–20:00, three to four hours after the maximum exterior temperature which is around 16:00, while the minimum temperatures inside the hut are around 08:00–09:00, two to three hours after the exterior minimum temperature which is around 06:00.

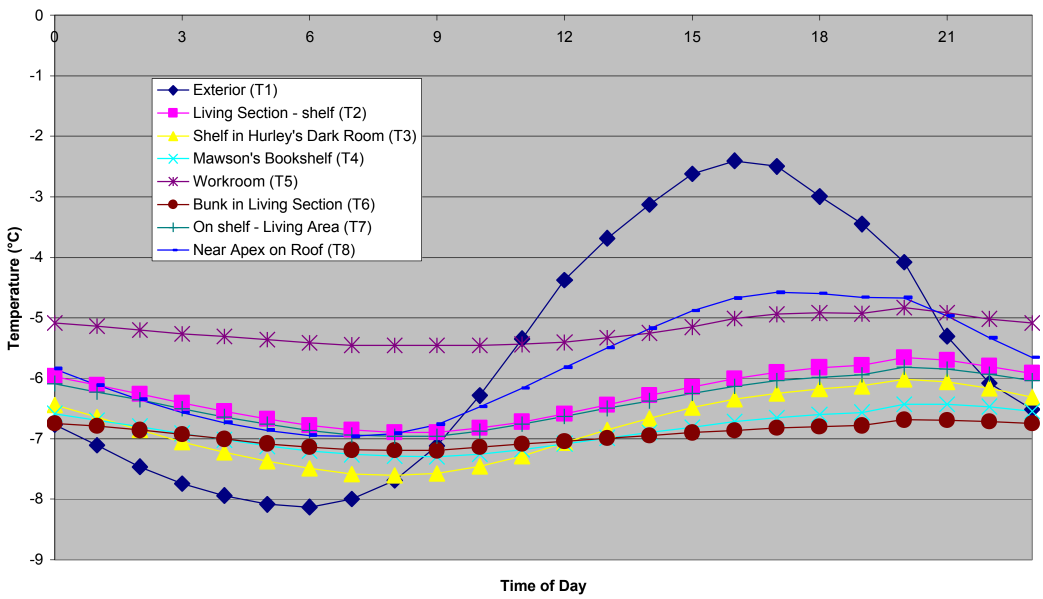


**Figure 4. Seasonal decomposition of temperatures, January to February 1999, Mawson's Hut**

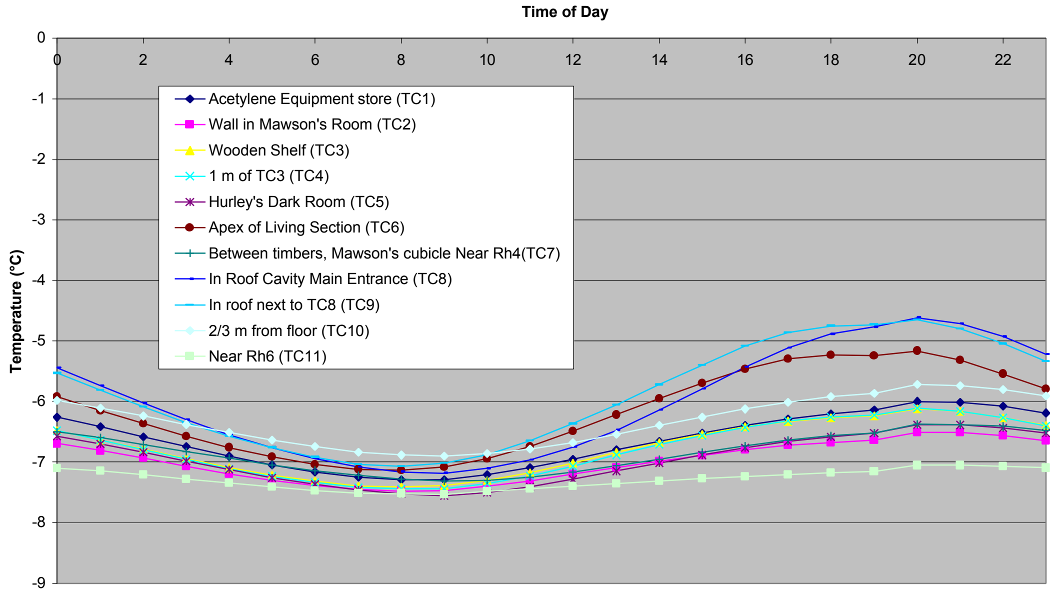


**Figure 5. Seasonal decomposition of thermocouple temperatures, January to February 1999, Mawson's Hut**

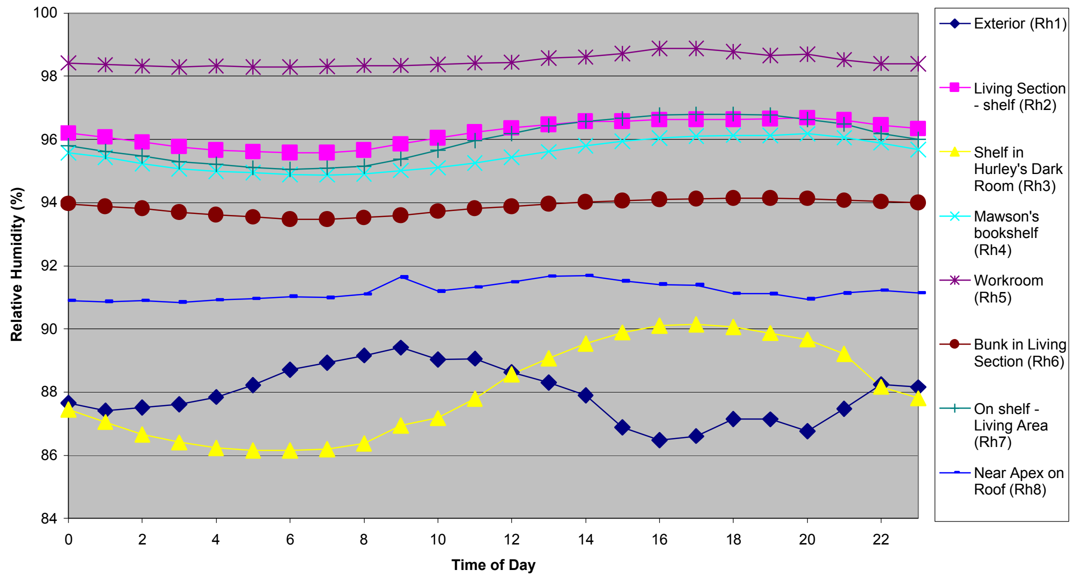


**Figure 6. Seasonal decomposition of RH, January to February 1999, Mawson's Hut**

The interior RH of the hut does not follow the exterior RH cycle. The interior RH is generally lowest when the exterior temperature is at its lowest (and the exterior RH is near its highest), and not when the interior temperature is at its lowest. The interior RH is at its highest an hour after the exterior temperature has reached its highest and three hours before the inside temperature has reached its highest. If the RH were only dependent on the temperature it would be highest when the temperature is the lowest and lowest when the temperature is the highest.

On examination of the absolute humidity graphs (Fig. 7), it is evident that there is a daily cycle of the mass of water in the air for all the locations. If the mass of water in the air is changing, there must be a source and sink for this moisture. Since there is more water present during the day and less at night, this implies that water is being evaporated during the day and condensed at night. For the water to be condensed there must be a structure/surface that is colder than the air.

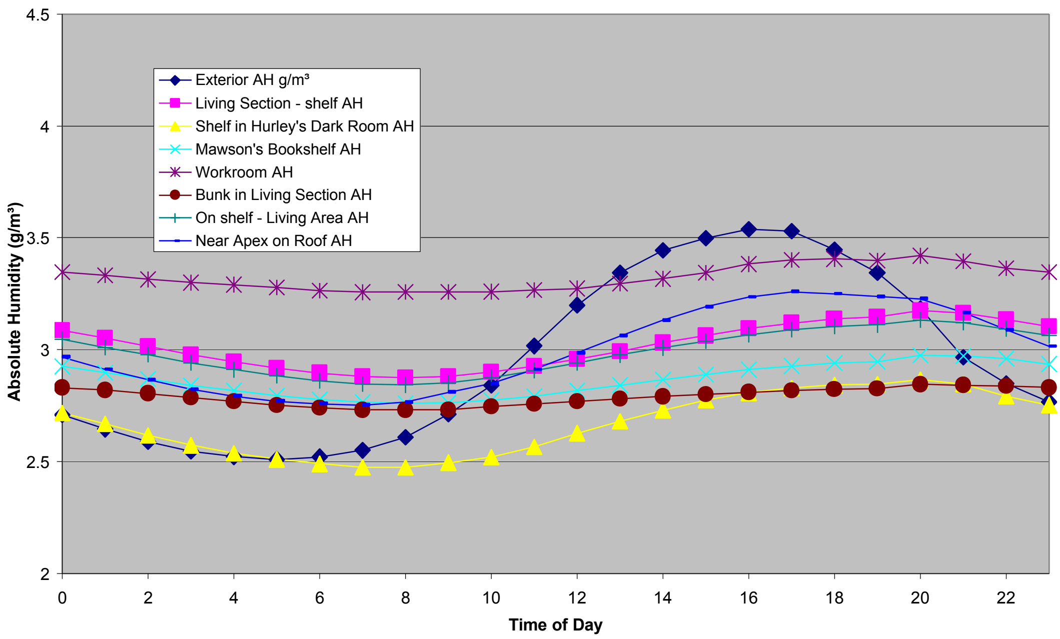


**Figure 7. Seasonal decomposition of absolute humidity, January to February 1999, Mawson's Hut**

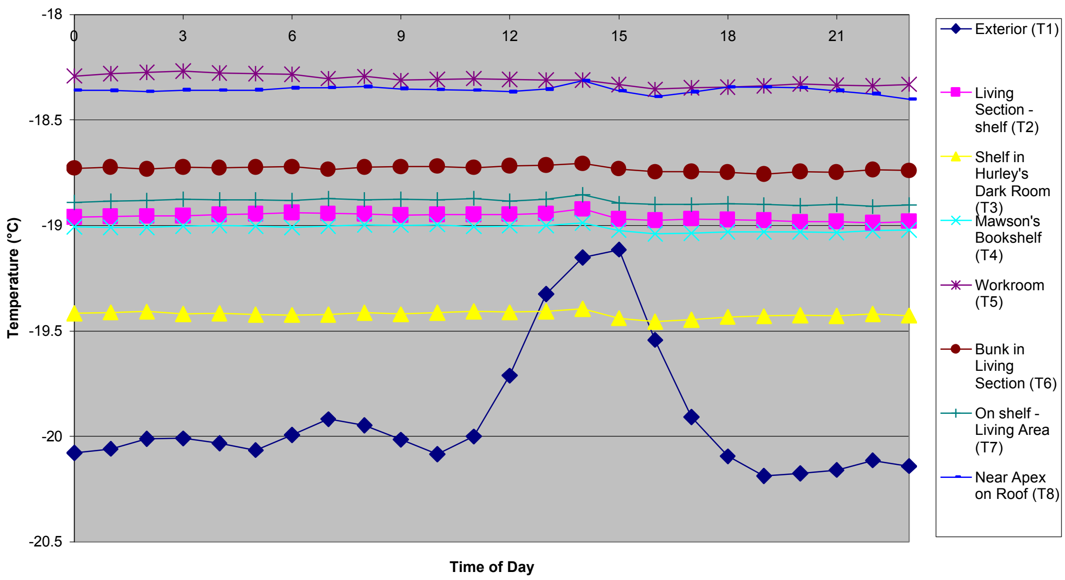


**Figure 8. Seasonal decomposition of temperature, May to August 1999, Mawson's Hut**

The observed presence of hoarfrost crystals on the timber inside the hut (Hughes 1988) is further evidence of condensation, as hoarfrost is formed when water vapour directly condenses to form ice. The timber acts as a suitable site for condensation by absorbing water vapour at night when the air temperature in the hut lags behind the exterior temperature and the timber is colder due to its connection to the exterior. When the timber warms up during the day the hoarfrost evaporates, thus increasing the moisture in the air.

Another possible sink for the moisture is the ice inside the hut. This would provide a cold surface on which more ice could form from the moisture in the air. Melt water accumulates around the exterior of the hut during summer, and it is evident from water stains on the building timbers that some of this water flows inside the building, which may provide a source of moisture.

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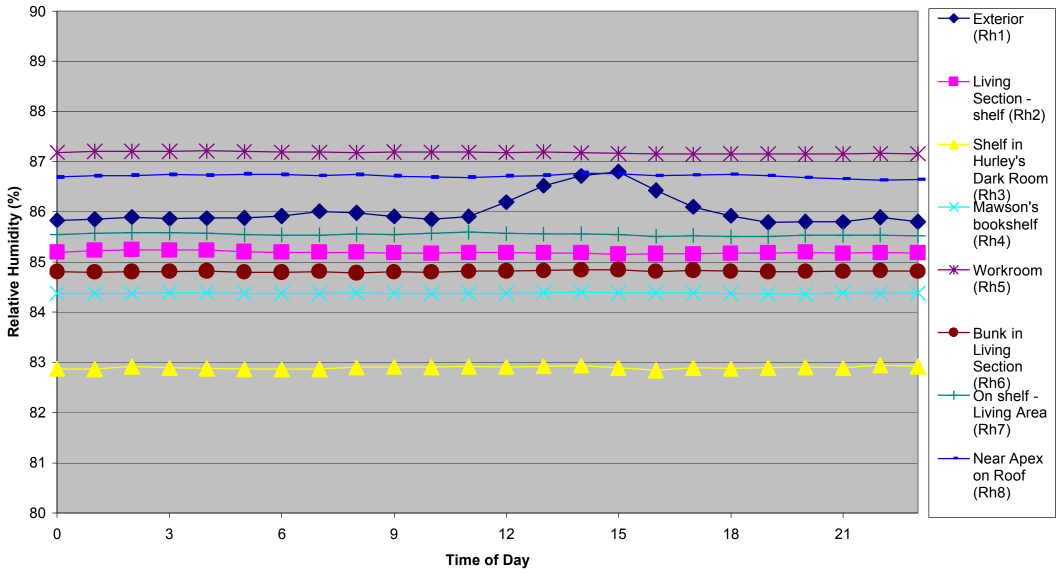


**Figure 9. Seasonal decomposition of RH, May to August 1999, Mawson's Hut**

During the winter period there is no significant daily cycle seen in any of the parameters or sensors. Only plots of the temperature and relative humidity are presented in this paper (Figs 8 and 9).

Despite the low temperatures in the hut, microbiological attack and corrosion have been observed. The data collected from monitoring reveals that there is plenty of moisture present for these two processes to occur. While it is initially surprising that these processes occur at low temperatures, the processes may occur slowly over a long period with significant intermissions during the particularly cold winter conditions. Even slow processes may become significant when considering the need to preserve historically significant buildings, where the aim is to preserve them for a much longer time than other buildings whose economic life may be quite short.

## 3 CONCLUSIONS

Despite the remoteness of the site and the difficulties of operating monitoring equipment in severe conditions, the results have shown that the data collected can be used to evaluate risks to the building such as corrosion and biodeterioration, with some limitations.

The data demonstrates that daily internal temperature and humidity cycles occur only when solar radiation is significant during summer. The interior temperature cycle lags the exterior by several hours and is less extreme, demonstrating that the current state of the building offers some insulating effect.

The high RH inside the hut produces a high moisture content in the timber that comprises the building and this may be sufficient to allow biodeterioration despite the low temperatures.

The calculated time of wetness using King's alternative temperature and relative humidity criteria gives a better correlation with observed corrosion than ISO 9223, and shows that corrosion is a significant risk to the building.

This paper demonstrates that data from monitoring of historic buildings provides information to enable an understanding of the processes causing deterioration. Monitoring is an essential tool that enables guidance before conservation treatments are carried out and analysis of their success.

## 4 ACKNOWLEDGMENTS

The authors acknowledge the invaluable assistance of Steve Martin from the State Library of New South Wales for the installation of the monitoring system, and Jim and Yvonne Claypole who assisted in the installation, downloaded the loggers and exposed corrosion coupons during their year-long stay at Cape Denison. The Australian Associated Press Mawson's Hut Foundation and the Antarctic Science Advisory Committee provided funding for the monitoring equipment, and the Australian Heritage Commission gave permission to install the monitoring system. George King, recently retired from CSIRO Building, Construction and Engineering, provided ongoing assistance with corrosion studies at the site. Rob Easther from Australian Antarctic Division and expedition members of the 2000/01 expedition to Mawson's Huts provided logistical and technical assistance. Don and Margie McIntyre assisted in shipping of equipment to Mawson's Hut. Steve Bailey and Campbell Scientific Australia assisted in the design of the instrumentation.

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# REACH - Development Of An Economic Appraisal Method For Cultural Heritage

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**Summary: The European Union funded REACH project was undertaken between May 1998 and May 2001 , and considered how economic analysis has been applied to Cultural Heritage and how this analysis might be improved. It took a cost/benefit approach to the subject and brought together willingness-to-pay studies that sought to establish the value that society places on actions to conserve heritage with longer established dose/response calculations based on models that relate environmental parameters and corrosion damage. A primary intention was that REACH would design and build a prototype management software tool to undertake cost benefit analysis. There was not sufficient data to provide a decision making tool for all situations - the necessary research to achieve this end will take many years to complete. What the software does is to provide a first demonstration of the type of calculation that is possible. A user friendly presentation of the existing material reviewed by REACH is incorporated into the software in order that the user will benefit directly from the work undertaken.**

**The model that REACH has developed in the course of its initial three years of research reflects the state of the art of air quality science and economic analysis in the field of damage to cultural heritage. This means that currently available techniques and results have been utilised. The REACH model is made up of a number of modules, each of which relates to an area of cost (or benefit) calculation. The level of sophistication of the models vary and the project includes a critical evaluation of their suitability. Without an integrated model, however, it would have been difficult to make any comparisons between different costs. Now that the model exists, it can be adapted in response to further developments in the field. It is designed in such a way that each component can be improved and substituted without the necessity to rebuild the entire framework**

**The first version of the model is complete and ready for demonstration. It has been decided to use a platform that has become very familiar to users, a browser based HTML document exactly like the familiar WWW pages to demonstrate the data gathered by the project, coupled with custom written software to actually undertake the cost-benefit calculations. Demonstrations of such calculations are provided based on case studies undertaken within the programme**

**Keywords: Heritage, Economics, Dose-response**

## 1 INTRODUCTION

The European Union funded REACH project was undertaken between May 1998 and May 2001, and considered how economic analysis has been applied to Cultural Heritage and how this analysis might be improved. It took a cost/benefit approach to the subject and brought together willingness-to-pay studies that sought to establish the value that society places on actions to conserve heritage with longer established dose/response calculations based on models that relate environmental parameters and corrosion damage. A primary intention was that REACH would design and build a prototype management software tool to undertake cost benefit analysis.

The REACH programme had three main objectives:

The development of a method to integrate the different aspects of cost/benefit analysis that can be applied to cultural heritage.

The development of a working prototype management tool with a modular design that can be used to evaluate cost/benefit scenarios at different scales.

The development and validation of the model and management tool by use of practical case studies.

This paper provides a summary of the findings of the project and the main findings. The full report will be available from the European Union in Brussels at a later date. The project was developed through the following topics (work packages):

- Air Pollution and Material Damage Functions
- Indirect Costs
- Direct Costs
- Policy Effects
- Management Tool

The project produced a series of reports which covered each of the five topics and in addition a series of case studies were completed at the following sites:

- Nidaros Cathedral, Trondheim, Norway
- Kristiania “Kvadraturen” , Oslo, Norway
- Prehistoric rock carvings, Tanum World Heritage Area and Bohuslän/Østfold, Sweden
- Municipal House, Prague, Czech Republic
- Jeronimos Monastery, Lisbon, Portugal
- Lincoln Cathedral, Lincoln, UK

# 2 DESIGNING A PROTOTYPE FOR UNDERTAKING COST/BENEFIT ANALYSIS OF EFFECT OF POLLUTION ON HERITAGE

## 2.1 Environmental factors

The project has incorporated GIS based regional data and software developed in earlier work and shown how specific calculations for individual buildings can be undertaken. The CorrCost calculation facility has been integrated into the REACH management tool, permitting users to calculate corrosion rates and associated costs at a scale appropriate to their individual uses. For monuments in rural areas the grid size can be large without reducing the quality of future calculations. In urban areas the grid size must be smaller to describe the local variation in the pollution parameters.

The data obtained were utilised in the development of the management tool. The deterioration and maintenace of monuments are strongly related to the local effects of environmental parameters. Modelling of the geographical distribution of these parameters is implemented in the CorrCost part of the management tool but imported as single figures at present into the REACH tool. Future development could see exapansion of the full cost benefit for area calculations.

## 2.2 Damage functions

A selection of dose-response/damage functions has been presented for a number of materials. The selection was based on the best available alternatives from a critical review of the literature. The functions were then reconciled for objects of cultural heritage and an assessment was made of limits and ranges of applicability of the equations. The final set of usable functions is given in Table 1 and these constitute the best general choice.

**Table 1 Set of usable damage functions.**

| Material | Damage function |
|---|---|
| Zinc and galvanised steel[a] | $t = [\ 0.14[SO_2]^{0.26}e^{0.021Rh}e^{f(T)}/R^{1.18} + 0.0041Rain[H^+]/R\ ]^{-1}$ |
| | $f(T) = 0.073(T-10)$ when $T \leq 10°C$, $-0.025(T-10)$ otherwise |
| Copper[b] | $t = [\ 0.00018[SO_2]^{0.34}[O_3]^{0.84}Rh^{1.06}e^{f(T)}/R^{1.06} + 0.0080(Rain[H^+]/R)^{0.93}\ ]^{-1/0.83}$ |
| | $f(T) = 0.028(T-10)$ when $T \leq 10°C$, $-0.054(T-10)$ otherwise |
| Limestone[c] | $t = [\ R/(2.7[SO_2]^{0.48}e^{-0.018T} + 0.019Rain[H^+])\ ]^{1/0.96}$ |
| Sandstone[c] and spongilit[c] | $t = [\ R/(2.0[SO_2]^{0.52}e^{f(T)} + 0.028Rain[H^+])\ ]^{1/0.91}$ |
| | $f(T) = 0$ when $T \leq 10°C$, $-0.013(T-10)$ otherwise |
| Brick masonry | $t = 70 \pm 30$ ($SO_2 \leq 10\ \mu g/m^3$), $65 \pm 30$ otherwise |
| Rendering | $t = 1000/(\ 15.5+0.124[SO_2]+0.013Rain[H^+]\ )$ |
| Bitumen felt | $t = 1000/(\ 47.7+0.327[SO_2]+0.080Rain[H^+]\ )$ |
| Concrete | $t = 50 \pm 30$ ($SO_2 \leq 10\ \mu g/m^3$), $40 \pm 30$ otherwise |
| Paints on steel | $t = [\ 5/(0.033[SO_2] + 0.013Rh + f(T) + 0.0013Rain[H^+])\ ]^{1/0.41}$ |
| | $f(T) = 0.015(T-11)$ when $T \leq 11°C$, $-0.15(T-11)$ otherwise |
| Paints on galvanised steel | $t = [\ 5/(0.0084[SO_2] + 0.015Rh + f(T) + 0.00082Rain)\ ]^{1/0.43}$ |
| | $f(T) = 0.040(T-10)$ when $T \leq 10°C$, $-0.064(T-10)$ otherwise |
| Paints on aluminium | $t = 1000/(\ 32.2+0.107[SO2]+0.027Rain[H+]\ )$ |
| Repainted aluminium | $t = 1000/(\ 62.9+0.37[SO2]+0.095Rain[H+]\ )$ |
| Paints on rendering | $t = 1000/(\ 18.8+0.278[SO_2]+0.070Rain[H^+]\ )$ |
| Paints on wood | $t = 1000/(\ 87.5+1.03[SO_2]+0.260Rain[H^+]\ )$ |

[a]R is equal to 20 µm for maintenance of galvanised sheet, 30 µm for replacement of galvanised sheet and wire and 60 µm for maintenance of galvanised profile.

[b]R is equal to 100 µm for copper roofing of 800 µm total thickness.

[c]R is equal to 5000 µm for large constructions and 1000 µm for ornaments and inscriptions.

The materials weathering steel, aluminium, bronze and glass are not included in the list since it has not been possible at the present stage to assign critical thickness values. It should be noted that for a particular object or situation other functions might be preferred. For example, all environmental parameters are not necessarily available and it may be more appropriate to use a simpler function. These functions were used in the final REACH software.

### 2.3 Methods for valuing cultural heritage

The projected reviewed the various non-market valuation techniques that can be used to estimate a monetary value for cultural heritage goods, both revealed and stated preference techniques. Most of the studies to date that have valued cultural heritage have used a stated preference technique. There are good reasons for this, directly related to the difficulties mentioned in applying revealed preference techniques to cultural heritage goods. In order to apply the revealed preference techniques, an observable decision must exist that reveals the individual’s value for the good, which is rare for cultural heritage goods. In contrast, the contingent valuation method and the other stated preference techniques are very flexible with regard to the goods they can value. The only limitation is that a credible scenario can be devised where the respondent can imagine making a trade-off between money and the level of the cultural heritage good.

Cultural heritage goods are also well suited to contingent valuation studies because most respondents accept the idea of public provision of these goods. Contingent valuation is also the only valuation method that can capture non-use values. The relative importance of non-use and use values generated by a cultural heritage good will vary from case to case, but it is clear that for at least some of these goods, non-use value will be very important.

There are also some unique challenges associated with applying contingent valuation to cultural heritage goods. An important issue is the sampling frame for the study – how large a population should be sampled? This question is important because a contingent valuation study can generate an over-focusing effect within respondents.

Finally, application of the contingent valuation method to cultural heritage goods can be made difficult due to the complexity of the issues involved in its provision. A typical programme to improve the quality of cultural heritage goods often includes both positive and negative components.

In general, economists prefer to rely on actual behaviour consumers undertake in response to price signals, as opposed to hypothetical statements about behaviour. The presumption is that actual decisions involving real money carry penalties for mistakes, and are therefore more reliable indicators of value. Unfortunately there are few good situations where observed behaviour reveals preference for cultural heritage goods. In those isolated situations, the presumed preference for revealed preference techniques applies to these goods as well as any other public good. However, stated preference techniques are often the only practicable approach to generating value estimates for cultural heritage goods.

Contingent valuation is especially useful in the situation where the good in question is a change in the characteristics of a building or monument, which is the type of heritage good which the REACH programme has focused on.

### 2.4 Environmental policy

In the case of the environmental (air quality) policy, most of the economic analysis available for review has examined all aspects of costs and benefits, with damage to heritage either a minor component or absent. The REACH policy workpackage has therefore highlighted the benefit of undertaking a specialist analysis of the cost to heritage. In general, as may be seen from the studies reviewed, the higher damages per ton of pollutant emitted belong to Central European countries, mostly because of the large population affected. France, Germany, The Netherlands, Belgium, and Northern Italy present very large damages. On the other hand, peripheral countries such as Spain, Portugal, Southern Italy or Ireland present much lower damages. The effects on heritage alone may well not reflect this general pattern entirely but there are some grounds for expecting that some features will be similar.

In spite of the general trend identified, it has to be noted that site specificity even within countries is still an important aspect due to the presence of large population centres near plant sites, the distribution of heritage sites and to the influence of background emissions. The influence of large cities is shown mainly for the waste incineration plants, which are usually placed near or in large cities. This location produces very large damages, as shown especially in the France, where particulates produce damages around 56,000 in the Paris area. These large damages per t of pollutant emitted require then that emission factors are kept to the lowest, so that the external costs of electricity generated by these plants are not excessive.

## 3 APPLICATION OF THE MODEL – A CASE STUDY EXAMPLE

The material presented in this Section focuses on the UK case study featured in the main REACH program – Lincoln Cathedral. During the course of the project information and data has been gathered about materials and costs at Lincoln Cathedral from the cathedral works committee, whilst the external environment has been monitored to assess the pollution loading and meteorological variables. The relevant parameters are summarised in Table 2.

**Table 2 Lincoln measurement parameters**

Materials – Masonry, lead and glass

| Pollution | 1970 | 2000 | Environment | 1970 | 2000 |
|---|---|---|---|---|---|
| Sulphur Dioxide (μg/m$^3$) | 100 | 10 | Rh (%) | 0.70 | 0.70 |
| Ozone (μg/m$^3$) | Unknown | 50 | Rainfall (mm) | 500 | 500 |
| $NO_2$ (μg/m$^3$) | Unknown | 20 | $H^+$ (mg/L) | 0.2 | 0.02 |
| | | | Temperature (ºC) | 10 | 10 |

The major materials on the cathedral are the stone, glass and lead. The glass is included for completeness but the areas of roof and wall necessarily dominate the calculations.

Two further pieces of information are required to determine the cost involved in replacing the materials – the amount of damage incurred in a time interval and the ‘acceptable’ damage limit with respect to the material and object. The amount of damage is determined by empirical damage equations (see Table 1), which vary from material to material and the damage limit is chosen in accordance with the location or use of the material.

For a general façade then an overall loss of 5000 μm is acceptable but for a delicate carving then a loss of 1000 μm might be more appropriate. Then from these damage limits the acceptable life time is 5000/D or 1000/D years.

For glass the leached layer depth (LL) is calculated from

$$LL = 0.013[SO_2]^{0.49}Rh^{2.8} \text{ nm per annum}$$

The technique for estimating this depth is fairly recent and the implication in terms of the deterioration of the physical features of the glass has yet to be resolved. Thus, no agreed critical depth is available. For the sake of demonstrating the calculations in this report a value of 10 nm is assumed.

The material damage equations for the REACH project are derived from a very large data set encompassing a variety of natural exposure conditions across Europe. Parameters and there ranges used in deriving the equations are given in Table 3.

**Table 3. Parameters and ranges used in deriving the equations**

| **Symbol** | **Description** | **Interval** | **Unit** |
|---|---|---|---|
| T | Temperature | 2 - 19 | ºC |
| Rh | Relative Humidity | 58 - 86 | % |
| [$SO_2$] | $SO_2$ concentration | 1 - 83 | µg/m$^3$ |
| [$NO_2$] | $NO_2$ concentration | 1 –121 | µg/m$^3$ |
| [$O_3$] | $O_3$ concentration | 14 - 82 | µg/m$^3$ |
| Rain | Rainfall | 327 - 2144 | mm |
| [$H^+$] | $H^+$ concentration | 0.0006 – 0.13 | mg/L |
| [Cl] | Cl concentration | 0.1 - 12 | mg/L |

Annual average values

There is no equation defining the deterioration of lead roofing. Thus, the presented calculations are for the masonry and glass only (Table 4.)

**Table 4. The material list for Lincoln Cathedral**

| Material | Area (m$^2$) | Cost (per m$^2$) | Damage Equation |
|---|---|---|---|
| | | | |
| Masonry restoration | 940 | 310 | $2.7[SO_2]^{0.48}e^{-0.018T}$ + $0.019Rain[H^+]$ µm per annum |
| Masonry Clean | 940 | 130 | Estimated values for illustration |
| Roof | 6,000 | 585 | No equation for lead |
| Aisle Roof | 2,700 | 560 | No equation for lead |
| Glass | 69 | 3000 | $0.013[SO_2]^{0.49}Rh^{2.8}$ leach depth nm. |

The restoration cost includes ancillary services such as scaffolding, surveyors, archivists etc. and a breakdown of the exact costs is not known.

Information relating to the value that people assign to say, cleaning or a particular restoration project, is particularly hard to obtain. Mostly this is achieved by visitor surveys which can be costly to design and implement. Conclusions from the economic studies carried out during the REACH project indicate that there is little correlation between different cultural heritage objects with regard to this willingness-to-pay, or other valuation techniques. That is to say, values derived at one location cannot be transferred to another location with any degree of certainty.

A limited survey undertaken at Lincoln as part of a different project indicates that a value of circa £20 per resident is not an unreasonable of willingness-to-pay for cleaning and restoration. Assuming that those who use the cathedral are willing to pay a little more then a willingness-to-pay value of £27 seems reasonable. The population of Lincoln is 80,000, the cathedral congregation perhaps 1000, and those that value the cathedral but never visit, say, 10,000.

In addition to these figures some form of estimate relating to the actual income streams is required. The budget of the cathedral with regard to conservation is some £2 million. Half of this is raise by grants and the rest by donation, there is also some income from renting surrounding property owned by the cathedral. The exact figures are not available and thus Table 5 is by means of an example.

**Table 5. Estimated income for different user categories**

| Category | Income (thousands) | % non-local |
|---|---|---|
| Users | 120 | 0 |
| Indirect Users | 5 | 10 |
| Non-users | 1 | 100 |
| Grants | 100 | 100 |
| Subsidies | 50 | 50 |
| Rent | 200 | 50 |

The % non-local figure indicates the percentage of the income that has its source from beyond the local region. That is to say it contributes to the local economy.

A key feature of the model is assigning the resulting spin-off income to Lincoln City in terms of the local or regional expenditure. The circulation of local money does not benefit the region; of greater interest is the monies bought into the region and retained there. This is also true of money spent working on the cathedral – the best scenario is that external money is spent on local work. The spin-off income can come from two sources, money from visitors attracted to the region or money spent by the workforce due to work carried out on the cathedral. Thus, in the data input there needs to be an indication of the source and sinks i.e. local or non-local.

### 3.1 Findings

For the environmental parameters and equations cited above we have the material lifetimes shown in Table 6.

**Table 6. Material lifetimes in past and present pollution climates.**

| | **Lifetime (Years)** | |
|---|---|---|
| **Material** | **1970** | **2000** |
| Limestone (carving/detail) | 45 | 103 |
| Limestone (Plain walling) | 222 | 516 |
| Glass | 219 | 481 |

The REACH management tool take these inputs and calculates :

- cost per annum over the life time of each material
- total cost arising from willingness-to-pay
- economic impact on the region from spin-off values arising from tourism & the workforce
- a comparison of cost between current environment and previous environment
- a comparison of cost between intervening prior to the end of material lifetimes

Applying these figures to the REACH management tool gives :

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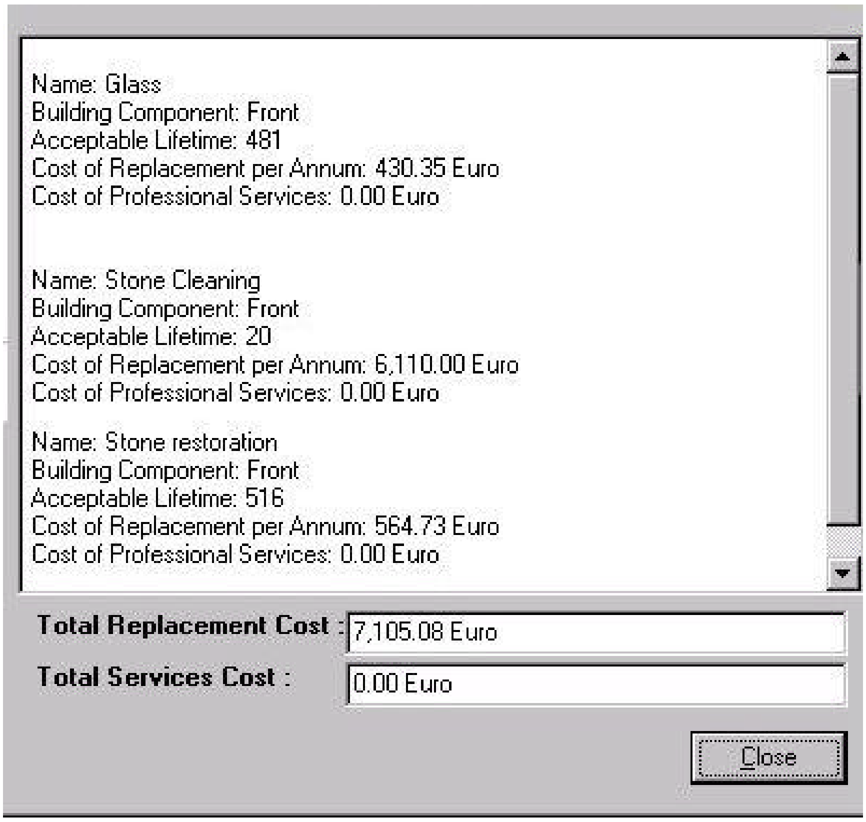


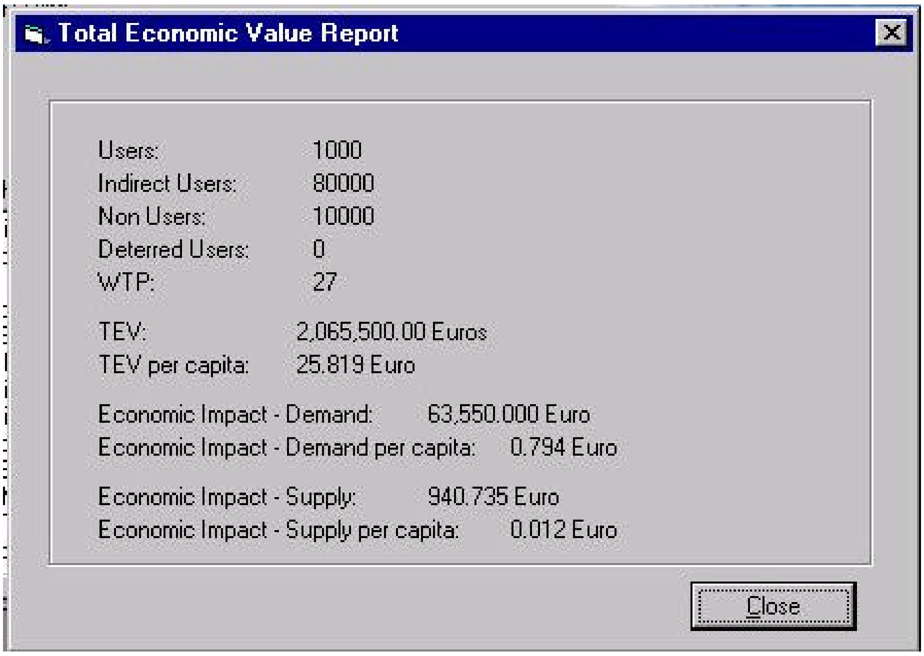

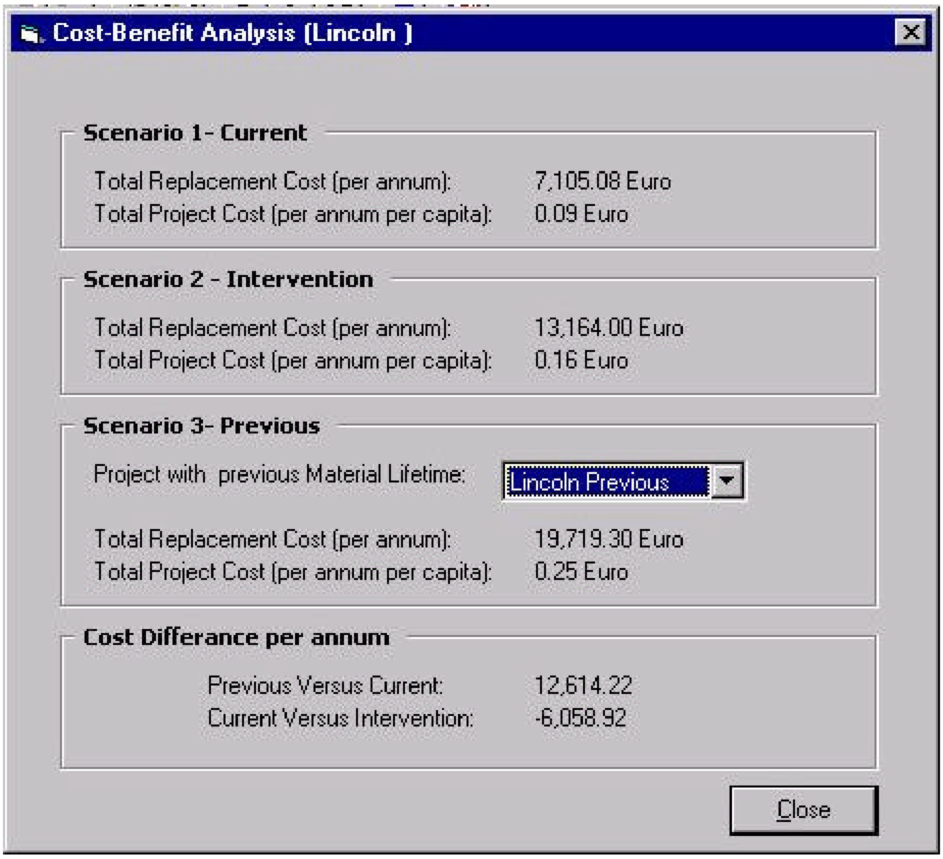


From this analysis we have that the combined cost for replacing the stone and the glass is dominated by the cost of the stone cleaning 7105 Euro per annum at a per capita cost of 0.09 Euro per annum for the population of Lincoln. This expenditure will result in spin-off income of 63,550 Euro per annum, at a per capita benefit of 0.79 Euro per annum for the population of Lincoln, due to tourist revenue and 940 Euro per annum, at a per capita benefit of 0.01 Euro per annum for the population of Lincoln, due to revenue arising from the workforce employed. The benefit of the latter revenue will of course be for the period of the restoration whereas the tourist benefit will be good for many more years than the duration of the restoration project.

This is in contrast to the willingness of people to see the cathedral restored valued at 2,065,500 Euro, a per capita cost of 25.8 Euro per annum for the population of Lincoln.

The intervention scenario is determined on the basis of replacing the glass after only 7 years and stone after 200 years – both times considerably shorter than the anticipated lifetimes of 481 and 516 years respectively. This shortening increases the cost per annum to 13,164 Euro per annum at a per capita cost of 0.16 Euro per annum for the population of Lincoln. Thus the net cost in reducing the materials effective lifetime is 6,058 Euro.

For comparison , the previous scenario, which is for the 1970s pollution climate, reduces the material lifetimes to 222 and 218 years for stone and glass respectively giving costs of cost per annum to 19,719 Euro per annum at a per capita cost of 0.25 Euro per annum for the population of Lincoln over 222 years. Thus the net cost saving in reducing the pollution with respect to the materials effective lifetime is 12,614 Euro.

## 4 HOW DAMAGE TO HERITAGE MIGHT BE REFLECTED IN FUTURE AQ POLICY

The intention of air quality legislation may be considered as either elimination or minimisation of the harmful effect of the pollution. It is generally accepted that elimination is an unreal objective; it would require zero atmospheric concentration and natural emissions ensure that this could never be met. Minimisation essentially means “reduction to a socially acceptable level”, which requires an assessment based on cost-benefit analysis and (for adverse health effects) medical analysis.

The abundant objects of cultural heritage create a precious treasure for the creation of a European identity. They play, however, also an important role in the development of tourism, which is of rapidly increasing economic importance. This was acknowledged already in the Treaty establishing the European Community, where the cultural heritage has been selected as a priority field action for the Community. Also the Council Conclusion (94/C 235/01) on drawing up a Community action plan in the field of cultural heritage envisaged specific actions for conservation and safeguarding of cultural heritage of European significance. The most efficient policy measure for safeguarding the objects of cultural heritage is a reduction of emissions of harmful pollutants. This approach is reflected i.a. in the Framework Directive on ambient air quality assessment and management (96/62/EC).

The importance of clean environment, which avoids occurrence of dirty buildings and severely damaged or destroyed monuments for the well being of people is generally recognised and accepted also by the World Health Organisation. This view is expressed in several documents issued during the years by the Community. In the Commission Recommendation to Member States concerning the protection of the architectural and natural heritage (75/65/EEC) it is stated that architectural and natural heritage is generally felt to be a determining factor in the quality of life. Also the Council Resolution on the integration of cultural aspects into Community actions (97/C 36/04) states i.a. the following principles:

- culture forms an integral part of Community action and contributes to the objectives of the Community through enhancement of citizenship and personal and human development,
- access to culture and the affirmation and expression of cultural identity are essential conditions for the full participation of citizens in society.

A confirmation that these declarations and principles have a firm acceptance among the citizens can be find in several studies of economic assessment of damage to cultural heritage, using the contingent valuation method. Respondents are asked a hypothetical question on their willingness to pay to preserve a certain historical building, which reveals their valuation.

To prevent or reduce harmful effects of pollutants on human health and the environment as a whole a Council Directive 1999/30/EC has recently been issued relating to limit values for sulphur dioxide, oxides of nitrogen, particulate matter and lead in ambient air. The limit values in the Directive have been established taking into account mainly the effects on human health, while for rural areas concern has also been given to protection of ecosystem and vegetation. The limit values for effects on materials have not been included even though the position papers prepared in connection with the preparation of the Council Directives states that the limit values for SO2 intended for protection of health and ecosystems may not be sufficient for preventing damage to cultural heritage. It was proposed, that Member States will be encouraged, where appropriate, to designate zones or areas where monuments need to be protected and where more stringent limit values, established by the Member State will apply. Limit values for some materials were proposed.

The non-inclusion of material limit values is a serious shortcoming for the sustainable protection and management of the European cultural heritage. The introduction to the air quality Directive states, that in order to facilitate the review of this Directive in 2003, the Commission and the Member States should encourage research into the effects of sulphur dioxide, nitrogen dioxide and oxides of nitrogen, particulate matter and lead. The pollution threshold levels for materials of cultural heritage objects are intended to be used in the future review of the European Air Quality Directive and thus directly contribute to an improvement of the legislation policy in this field.

The USA distinguishes between primary air quality standards (to protect public health) and secondary air quality standards (to protect against decreased visibility and damage to animals, crops, vegetation and buildings). The rationale for this distinction is that the dose-response functions for some pollutants may show a threshold for health effects but a linear relationship without threshold for monument damage; also that biological systems can naturally repair damage but monuments can not. For example, the USEPA has set a secondary 3 – hour average National Ambient Air Quality Standard (NAAQS) for sulphur dioxide.

Within Europe, only primary standards have been established. However, the work of REACH has provided additional information on dose-response relationships, willingness-to-pay and cost-benefit analysis; information that is now available to policy makers, for example through the Clean Air for Europe (CAFE) initiative. This will support the scientific basis on which the question of whether or not to introduce secondary air quality standards in Europe can be debated.

Results from studies of economic impact of pollution on objects of cultural heritage using the tool developed in the present project may facilitate the efforts to take into account material limits in the future revision of the air quality directives.

## 5 OVERALL CONCLUSIONS

The use of the REACH management tool, which is designed to estimate the costs and benefits of keeping our cultural heritage in good condition depends on the input of valid data and results from models, some of which already existed at the start of the project. The work packages in the project were selected to review the existing capability, to identify gaps and to examine how the particular problems associated with care for heritage might be incorporated.

The environmental effect to historic buildings and sites can be modelled today on different scales. In Europe the EMEP data base and models can give a good estimation of the environmental parameters on a European scale. For local areas, such as a city, several models based on interpolations between measurements or from emission inventories can be used. Still there are problems to be solved when the models change from a local scale to a European one or the other way.

All types of degradation depend on what is happening on the surface at a micro-scale. Modelling of the micro level impact of environmental factors is complicated and no complete model exists today. However the local models can often be used with good results since the damage functions are obtained by used the same environmental data.

The environmental impact is used to estimate of the lifetime and corrosion rates for materials. The dose-response and damage functions developed currently available describe mostly the meteorological and $SO_2$–pollution impact on materials. The “modern” environmental impact from a multi-pollution situation where $SO_2$ concentration is low while $NO_2$, $O_3$ and particle concentrations are still high, is not described in these equations. Damage functions that describe the time between maintenance

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and repair are the most important equations for calculation of cost for materials. The most reliable equations are the recent equations obtained for dose-response functions. To transfer these equations to damage functions we need to have values for how much corrosion loss can be accepted before repair must be done. This has been done for zinc, copper and calcareous stones. For many of the other materials very simple equations based on inspection of real buildings are used. Historic buildings are often built of materials where little is known about the environmental impact. For example stones like marble, granite and soapstone have no damage functions attached. Too little is known about important new materials like concrete.

To calculate the cost we need to know the amount of material exposed. For a single historic building the best way normally will be to measure the surface area either on the building itself or from drawings. If it is desirable to calculate the cost for a large group of houses like for a city a statistical approach is more useful. Statistical values for different house types have been obtained in several countries such as Sweden, Norway, England, Germany and Czech Republic. For parts of Europe these data sets can be applied but particularly for the Mediterranean countries statistical date for use of materials are needed.

In most counties contractors dealing with repair of buildings have price lists for estimating the cost for the work they have to carry out. To estimate the cost for repair and maintenance of historic buildings has been more complicated to obtain. More reliable data for repair and maintenance cost has been obtained through some of the REACH case studies.

To calculate the total social costs of corrosion we need to add the welfare loss to visitors and non-visitors from experiencing the corrosion damages to the cultural heritage objects, i.e. their loss in well-being from knowing that cultural heritage object get damaged. This welfare loss can be measured in terms of the willingness-to-pay (WTP) of visitors and non-visitors. These costs are often termed indirect costs and should be included in social cost-benefit analysis (CBA) of e.g. restoring historic buildings. CBAs assume a national level of decision making. If the level of decision-making is local, e.g. a community, welfare losses to households in the community only should be counted, but also all direct losses in local income and employment and indirect (multiplier) effects on other sectors of the local community.  These indirect costs, both at the national and local level, could make up a substantial part of the total costs.

## 6 ACKNOWLEDGMENTS

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# The Historic Belfast Timber Truss - A Way To Promote Sustainable Roof Construction

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**Summary: Historically, timber has been used extensively for roof structures. In the 19th century demand for clear span industrial buildings brought about the development of a variety of timber truss types. The 'Belfast' truss was developed circa 1860 to meet the demand for efficient wide span industrial buildings. This is essentially a bow-string configuration with a curved top chord, straight horizontal bottom chord and close-spaced lattice bracing. It is known that several thousand still exist in Ireland, many in buildings of historic significance. Although the manufacturers claimed superior durability for the trusses, degradation has occurred mainly as a result of insect infestation and wet rotting fungi.**

**This paper seeks to demonstrate the efficiency of the Belfast truss and to establish that, by modern structural analysis techniques, trusses can be replicated in historic buildings almost exactly as the original. Results of a theoretical study have been compared with the behaviour of two full-scale trusses: one as a replacement truss, tested in the laboratory; the other an 80-year-old truss tested on site. In addition, experimental results from a manufacturer's archive material of full-scale truss tests carried out about 100 years ago have been compared with theoretical models. As well as their significance in historic building conservation the paper proposes that Belfast trusses are an attractive sustainable alternative to other roof structures.**

**Keywords: Timber, trusses, roofs, historic buildings.**

## 1 INTRODUCTION

A booklet produced in 1930 by Anderson & Son to promote the 'Belfast' truss begins "The curved lattice girder or 'Belfast' roof, needs no introduction, having been in use for over half a century. Its advantages for the purpose of roofing wide spans, without intermediate supports, are well known." Indeed timber trusses of this type appear to have been in use from about 1866, fabricated by McTear & Co., Belfast (Gould et al. 1992). While its origin is somewhat uncertain, the efficiency of its structural behaviour was widely appreciated and the truss was employed extensively throughout Ireland and Great Britain on industrial buildings, farm buildings, airfield hangars etc. for spans ranging from 6 m to 36 m. The usage diminished around the 1930s, although farther afield it is known that trusses were regularly made in Lagos until the 1950s. Several hundred buildings with Belfast trusses are extant in Ireland; some 70 locations are known in Northern Ireland alone and the list is not exhaustive. The authors have been involved in the analysis, repair and replication of trusses on several buildings - most recently on the replacement roof of a farm building. In this case the existing trusses, severely degraded by woodworm infestation were replaced with new trusses having the same profile but a simplified internal configuration. An extra truss was fabricated for this project and tested under laboratory conditions. A larger span (12 m) truss in an existing warehouse has been tested to destruction on site. Results of ultimate load tests carried out on prototype trusses in 1906 by Andersons have been found in the company's archives. The analysis, design, fabrication and testing of trusses have resulted in a better understanding of their behaviour which is not only of historic interest and fundamental to the repair/restoration of existing trusses, but also relevant to the design of modern timber trusses and the promotion of a sustainable form of roof construction.

## 2 HISTORICAL BACKGROUND

Timber truss forms have been used since ancient times and enable spans that are longer than the pieces of timber available. Trusses were utilised by Palladio and Wren for major roofs, employed extensively in N.America where the material was readily available and by the beginning of the 19th century their behaviour was well understood. In the first half of that century 'king-post' and 'queen-post' trusses were widely used for industrial buildings, typically for spans to 9 m and 18 m respectively. Another truss form, the 'hammer-beam', which did not require a tie at eaves level, would have been an expensive alternative in these situations. Many of these trusses survive and are still economical to fabricate, particularly the king-post type, without the need for special mechanical connector devices. The 'Belfast' truss was developed to meet the demand for efficient and larger span roofs, brought about by the industrial revolution. It is no coincidence that Belfast was a centre for the production of felt, a material eminently suited as a lightweight weatherproofing membrane on the curved roofs. The Treatise by Newlands (1860) on timber construction makes no mention of the Belfast truss, so it might be assumed that the reference to the McTear truss of 1866 indicates a reasonably accurate date of origin. At the beginning of the 20th century new building types for shipbuilding and aircraft created a demand for even wider span roofs. Figure 1 shows one of the many Belfast truss roofs at the Harland and Wolff shipyard, Belfast, incidentally adjoining the slipway on which the 'Titanic' was later built. The photograph, taken in 1899, shows a set of trusses spanning approximately 24m supporting purlins and sarking boards. Construction of an aeroplane hangar in 1918 with a two-bay stepped roof, is illustrated in Fig. 2. In Fig. 3 a Hanley Page 'bomber' is shown outside the finished building where parallel chord steel trusses support the timber trusses over the hangar doors.

**Figure 1. Harland&Wolff shipyard store 1899**
**(©UFTM no.H590)**

**Figure 2. Aeroplane hangar construction Aldergrove 1918 (©UFTM no.H2451)**

## 3 TRUSS CONFIGURATION AND BEHAVIOUR

The general arrangement of the truss is shown in Fig. 4, consisting of a two-piece bowed top chord, a two-piece horizontal (or slightly cambered) bottom chord between which are sandwiched and nailed lattice (bracing) members. The truss profile is very efficient for uniformly distributed loading - it behaves in essence as a tied arch, with the thrust line almost coinciding with the alignment of the top chord, resulting in very small forces in the lattice members.

Various lattice layouts have been used since the trusses were first developed, all based on uniform purlin spacing along the top chord, as shown in Fig. 5 (a) to (d). It is thought that these arrangements were adopted to facilitate fabrication. For example, the pattern in Fig. 5 (d), is the most convenient to establish within a limited working area.

**Figure 3. Hangar of Fig 2 as finished**
**(© UFTM no.455)**

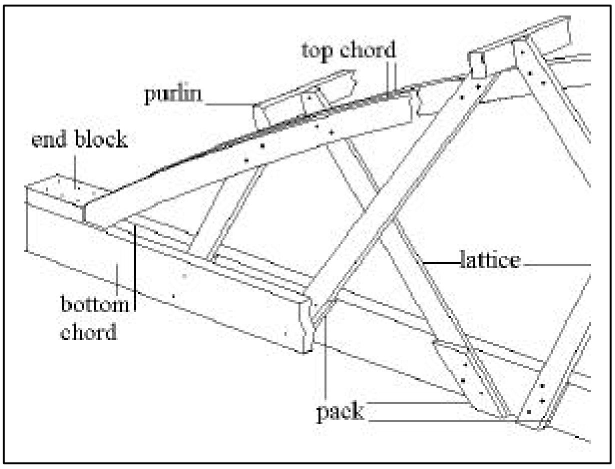


**Figure 4. Belfast truss–general arrangement**

The eaves joint, connecting the top and bottom chords, is the most critical detail in the truss. Figure 4 shows an 'end block' which 'stops' the horizontal component of the compression force in the top chord and transfers it as a tension force to the bottom chord. This arrangement is similar in concept to the eaves detail on a 'king' or 'queen' post truss. The block was not used on all the trusses examined and where it was present, the nail fixing to the bottom chord did not always appear to be effective.

The trusses were made both on and off site. It is probable that the bigger trusses were made on site. Figure 6 is a good general view of Belfast trusses being fabricated - these are thought to be the same trusses shown in the hangar building of Figs. 2 and 3. It is interesting to note, that the eaves joint detail here differs from that sketched in Fig. 4. In the right foreground of Fig. 6 the lattice members form a solid gusset at the eaves joint and double members are used in the region close to the eaves. This obviously improves the load capacity of the joint and a possible reason for the double members is discussed later.

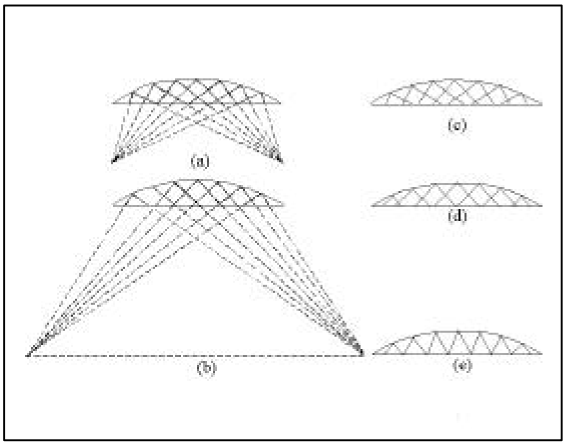


**Figure 5. Variety of lattice layouts**

**Figure 6. Fabrication of Belfast trusses c1917**
**(© UFTM no.426)**

## 4 DURABILITY ASPECTS

In all the trusses examined there appears to have been no timber treatment by creosote, solignum or the like. However, nearly all the trusses used naturally durable timber species, namely European redwood (*Pinus sylvestris*) and American pitch pine. In some trusses spruce was used for the lattice members and as a result often showed attack by beetle. The most common wood-boring insect in Ireland is the furniture beetle (*Anobium punctatum*) whose larvae together with those of other beetles, are known as 'woodworm.' This insect is responsible for most of the infestation in Irish buildings. Although damage is generally confined to sapwood, it can extend right through timber, such as spruce, within which there is no clear differentiation between sapwood and heartwood (Gilfillan and Gilbert 2001).

In relation to the felt roof finish, the Anderson Company claimed that "the original covering would last up to 40 years." This may have proved to be optimistic and there is evidence that ingress of moisture has caused 'wet rot' in some trusses, particularly at the eaves where the water was directed off the curved roof. The durability of this connection was also influenced by the type of gutter detail used. In Fig. 7(a) the gutter is located clear of the roof whereas in Fig. 7(b) it can be seen that any breakdown in the gutter lining will allow water to penetrate into the end region of the truss. The authors have designed repairs for eaves connections on two of the trusses in the aeroplane hangar roof shown in Fig. 2, in which the gutter detail is inside the building. The repairs took the form of steel splice plates to re-establish the structural connection between the top and bottom chords. The warehouse truss recently tested (truss no.3 in Table 1) was one of a set with the simple external gutter detail. None of these trusses showed any sign of degradation at the eaves detail.

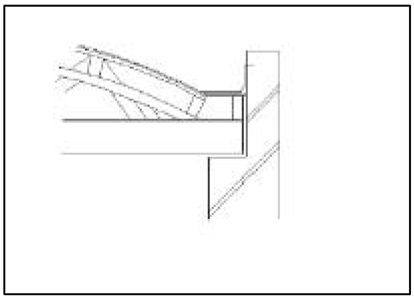

**Figure 7(a). Gutter-external**

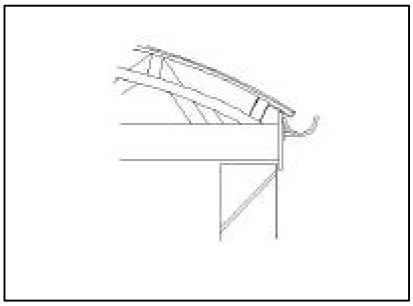

**Figure 7(b). Gutter-internal**

## 5 THEORETICAL MODELLING AND EXPERIMENTAL TESTING

Table 1 summarises the dimensions and behaviour of six trusses modelled theoretically. Three of these were subsequently tested to failure.

Modelling was carried out using a plane frame analysis software package. Two web layouts were examined for the warehouse truss, namely layouts (a) and (b) shown in Fig. 5. These are denoted as truss nos. 3 and 4 in Table 1. In fact, for the larger span trusses, it was very difficult to establish on site which web layout had been used. For each analysis the lattice members

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were modelled with simple pin end joints while the eaves joint was considered both pinned and fixed, the latter condition corresponding to the use of a gusset plate.

The three experimental tests have been mentioned in the introduction: truss no. 1 was a new 'bow-string' fabricated to replace truss no. 2 (the original trusses could not be tested due to severe woodworm infestation); truss no. 3 was the existing warehouse truss, tested in place using hydraulic jacks and a beam arrangement anchored to a tracked excavator; and truss no. 5 was the Anderson prototype truss for which a test report, dated 1906, was available from the company. Unfortunately, only ultimate load was recorded for the test on truss no. 5, whereas it was possible to monitor deflections in the other two tests.

**Table 1. Truss dimensions, modelling and testing summary.**

| Truss no* | Span L(mm) | Height h(mm) | L/h | Top chord radius (mm) | Model defln. (mm) | Expt. defln. (mm) | Model failure load (kN) | Expt. failure load (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | 6600 | 623 | 10.6 | 9060 | 12.7 | 10.8 | 26 | 23 |
| 2 | 6600 | 623 | 10.6 | 9060 | 12.6 | - | 26 | - |
| 3 | 12000 | 1300 | 9.2 | 14500 | 14.9 | 18.0 | 45 | 28 |
| 4 | 12000 | 1300 | 9.2 | 14500 | 14.6 | - | 45 | - |
| 5 | 11000 | 1200 | 9.2 | 13200 | 13.7 | - | 119 | 67 |
| 6 | 11600 | 1260 | 9.2 | 13980 | 14.4 | - | 91 | - |

* truss type/lattice layout:

1. new bow-string (Fig. 5(e))
2. truss replaced by no. 1 - 'fan' (Fig. 5(a))
3. warehouse - 'fan' (Fig. 5(a))
4. warehouse - 'right angle in semi-circle' (Fig. 5(b))
5. Anderson prototype test - 'right angle on top chord' (Fig. 5(c))
6. workshop/factory - 'right angle on bottom chord' (Fig. 5(d))

# 6 DISCUSSION

## 6.1 Truss configuration.

The Belfast truss profile is clearly efficient for uniformly distributed loading. Analysis showed that even for non-uniform loading, which might be caused by snow, the lattice forces remain relatively small. The connections of the lattice to the top and bottom chords are not subject to large forces and secondary bending due to non-concurrence at these joints is not significant.

In terms of the variety of lattice layouts outlined in Fig. 5 analysis showed that these alternatives make little or no difference to all the member forces.

The most common fabrication method produced initial lack of straightness and induced bending stresses in the lattice members. These were required to bend when crossing adjoining members because, to accommodate the lattice, the bottom chord pieces were separated by twice the member thickness and the top chord pieces by only the member thickness. The consequent lateral displacement is equivalent to half the lattice member thickness, producing a critical bending stress condition for the shorter members and a critical instability condition in the longer members. Later trusses incorporated double lattice members close to the ends, as seen in Figs.2 and 6, to compensate for the incipient weakness in the lattice.

## 6.2 Truss behaviour.

The top chord was undersized relative to the bottom chord for all the trusses analysed. However, it is thought that the tightly jointed sarking board (typically 16 mm thick), forming the roof deck, supplements the top chord by composite action. In addition, this curved deck may act independently as a barrel shell. There is no doubt that the sarking and purlins also prevent instability of the top chord. This was evident in the difference in the failure modes between the laboratory-based load test (truss no. 1) and the on-site tests. Thus these supplementary effects, composite action and lateral restraint, allow a smaller top chord section size, which facilitates fabrication.

Modelling predicts the presence of bending stresses in the bottom chord concentrated in the length between the eaves joint and the quarter-span point. These stresses become greater as the truss becomes more shallow. It is interesting to note that, in Table 1, all the larger span trusses have a relatively high span to height ratio of 9.2. Generally the ratio ranges from 5 to 10. The higher ratios also produce a more acute angle at the eaves, increasing the connection force there. For 'modern' bow-string trusses it is convenient to set the radius of the top chord equal to the span dimension. This results in a span/height ratio of approximately 8.

### 6.3 Experimental testing and theoretical modelling.

The warehouse truss (no. 3), tested in place, was slightly less stiff than predicted, as shown in Table 1. Ultimate failure was caused by slippage of the eaves joint and buckling of several lattice members close to the quarter-span position. An end part of the truss was cut out after failure and the timber, redwood chord members and spruce lattice members, was found to be in sound condition. Joint movement, especially at the eaves, could explain the reduction in stiffness.

The three pairs of trusses tested by Andersons in 1906 and observed by McKenzie and Young, Architects and Civil Engineers, all 'gave way' in the bottom chord near the quarter span position. Unfortunately, the type of failure is not reported, deflection readings are not given and the detail of the eaves joint is not illustrated. The analysis would not predict this failure. However, since the trusses were tested in pairs with the roof covering in place it may be that the behaviour was enhanced by composite action and the stabilising effect on the top chord.

Truss no. 1, laboratory tested, was one of a new set fabricated to replace existing Belfast trusses (truss no. 2). The outline of the original trusses was not changed since there was a requirement to match an existing roof in terms of span and radius. However the internal lattice, was substituted by a more conventional 'bow-string' configuration, similar to that shown in Fig. 5(e). The measured stiffness of the truss was slightly higher than that predicted. This could be explained by some fixity at the lattice member joints. The truss failed, by buckling in the top chord, at an ultimate load approximately 10% lower than predicted.

This truss was compared analytically with the original Belfast truss of the same outline. It was found that there was little difference in truss member forces except that the bow-string truss showed lower bending moments in the bottom chord and lower axial forces in the lattice members.

### 6.4 Fabrication and efficiency.

In terms of fabrication the bow-string truss can be compared with the Belfast truss:

(i) the lattice layout geometry is simpler;

(ii) the fabrication is easier in terms of joints - the lattice members do not cross over each other and both chords have the same separation between the pieces to accommodate the lattice members;

(iii) the lattice members are straight - there is no initial bending and incipient instability;

(iv) the detail of the eaves joint is simple and effective - a plywood gusset of the same thickness as the lattice members could be used.

In terms of efficiency it is interesting to compare these trusses with beams. Considering the case of trusses no.1 and no.2 the volume of timber used is almost the same. Nearly four times that volume of timber would be required to produce a beam with the same load carrying capacity as the trusses and such a section, approximately 350 mm deep, might need to be glue laminated.

## 7 CONCLUSIONS

1. The Belfast truss is a remarkably efficient structure for wide span roof construction.
2. Historically, many geometrical layouts have been used for the internal lattice members, but these variations make little difference to the truss behaviour.
3. The span to height ratio, which depends on the top chord radius used for a particular span, is the most important influence on the truss behaviour. It is recommended that the ratio should be about 8, resulting from a chord radius equal to the span dimension.
4. Experimental testing provides convincing evidence that the roof decking enhances the truss behaviour by composite action and the provision of lateral restraint to the top chord.
5. The trusses in many historic buildings have suffered from the effects of water penetration due to failure of the felt covering and leakage from the gutter. Modern high performance membranes and gutter linings will help eliminate this problem.
6. In many historic buildings the eaves connection, even in sound original condition, is inadequate. The provision of a solid boarded or plywood gusset represents a superior connection design.

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7. The bow-string truss with simple lattice layout has a fabrication advantage over the traditional Belfast types. The bow-string truss is however only slightly superior in performance to the earlier types.

8. In the case of historic buildings, where the replacement of Belfast trusses is necessary, it is perfectly acceptable to replicate the original truss, subject to the provision that the eaves details may need to be altered. These are the most critical joints in the truss and a gusset-type connection may be needed to transfer the horizontal component of the top chord compression force into the bottom chord.

9. In terms of timber volume, all the trusses reviewed are considerably more efficient than equivalent beam structures. However, the superior efficiency of trusses relative to material used must be offset against higher fabrication costs. Nevertheless, these trusses can be fabricated without expensive equipment and specialist skills. The Belfast truss represents an elegant and efficient structure, which can utilise renewable, even home-grown, material economically.

## 8 ACKNOWLEDGEMENTS

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# A Study Of IT Developments In The Next 10 Years And Their Influence On Classification In Building

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**Summary: As part of a large collaboration project in Denmark on improving classification systems used in building, a research study is being carried out on the influence of developing technologies. This takes five areas for application of IT that are particularly relevant to classification of data, and uses groups of experts to set up scenarios for their capabilities and conditions for success in the next 10 years. These are then presented in a way which non-technical people can understand, and a sample of firms from all parts of the Danish construction industry are asked whether they would be likely to take them up in that period. This methodology should avoid the overenthusiastic bias, which some other studies have had, towards technology and give a realistic and balanced view, taking into account the process changes and investments necessary in companies of all sizes.**

**Techniques used for studies of future IT usage have used maturity modelling to analyse the co-maturity of business processes and IT development. This experience has informed the current study and the summary tables, included in this paper, on the views on technology given in the workshops are being converted to a more widely understandable form and these, and the opinions of Danish construction industry firms, will be reported at the end of 2001. The results of the IT barometer survey for Denmark, some of which are included, will also be compared with Finland and Sweden.**

**The data from the two studies will be used to estimate the level of future use of innovative systems and their need for, and influence on, standard forms of classification. The information collected will be combined with the work of other groups in the classification project to test and integrate new systems being proposed for organising building elements, product data and schedules of rates. The scenarios are being tested in autumn 2001 with the final report due at the end of the year. The results will be included in the presentation of this paper.**

**Keywords: Information technology, classification, maturity models, IT futures**

## 1 INTRODUCTION

The Danish Centerkontrakt 'Classification in building' is a three-year project supported by the Ministry of Industry and coordinated by the Teknologisk Institut. The Technical University of Denmark, BYG.DTU group on Construction Management, is contributing research on international experience of classifying building elements, products and prices, and studying future effects of new developments in IT. A final recommendation for a new classification system for the Danish building industry, growing out of the currently used SfB system, is due at the end of 2002.

The DTU research project finishes at the end of 2001 and the objective is to ensure that a Danish system is compatible with other systems used in Europe, and with IT systems expected to come into general use within the next ten years. New developments in building modelling, e-business and mobile communications are likely to have a major effect on the way building information is organised. A methodology was established to obtain a realistic view of which systems are likely to be taken up in design, construction and building management. The final analysis of responses to the series of future scenarios summarised in this paper is not due until late 2001.

There are many methods for trying to predict future developments in IT, but it is the likely take-up of the tools becoming available which is of particular interest for this project. A number of IT futures studies have been carried out for the construction industry, particularly in the UK, and the methods used have been compared and summarised (Howard 2000). The five methods compared were:

- Analysis of historical data and projections from this into the future
- Surveying the views of a typical sample from industry
- Polling experts on the likely success of future scenarios
- Presenting headlines on possible future events
- Multimedia visions of possible future scenarios

In the current research, the first and second of these methods were used together with future scenarios defined with the help of experts. In order to temper the enthusiasm of experts for their favourite technologies, an additional stage is added which involves presenting the scenarios in a manner suitable for non-experts to comment upon, and asking a sample from the Danish construction industry whether they were likely to become users of the technologies expected to become available to them.

Five areas with particular relevance to classification were chosen for these scenarios:

- Shared project information
- Building modelling
- Product information
- E-business
- Standard descriptions

Data from a repeated survey of IT use in construction was used to provide some indication of growth rates and future expectations (Howard 1998, Howard 2001). A series of workshops were held at DTU involving small groups of experts on IT and various areas of construction, and the initial scenarios, presented in this paper, were defined with their help before being presented for wider feedback.

Availability of a new technology is only one factor affecting its usage. Where it supplants existing techniques, as CAD replaces manual drawing or mobile communications can provide access to information anywhere on a building site, growth is relatively easy to predict. Where process change is required in order to obtain the full benefit of a technology, prediction becomes more difficult and may depend upon developments across the whole industry. Maturity models offer one approach to compare the stage of development of a technology with the likely benefits to a whole industry (Lautanala et al 1998), or with the levels of maturity in each stage of the construction process (Aouad et al 1998). The second of these studies uses a process representation and 'spider web' diagrams from (Karandikar et al 1992) to present levels of maturity in both process and IT, 'Fig 1'. The problem is always to link particular IT facilities with particular process stages. If this can be done, it indicates whether a company, or a whole industry, is ready to take up the relevant IT development. This is much easier for a specific company and there are methods, such as the Strategic Health Check (Construct IT 1999), for assessing levels of maturity.

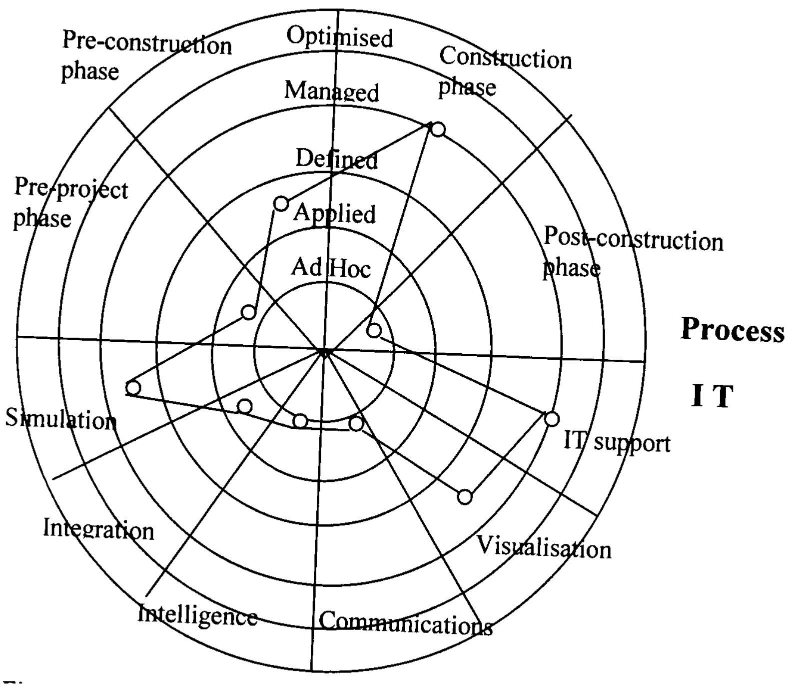


**Figure 1. Diagram showing maturity levels in processes and technologies. (Aouad et al 1998)**

## 2 THE IT BAROMETER SURVEYS

Surveys of the levels of use of IT in the construction industries of Denmark, Finland and Sweden were carried out in parallel in 1998 (Howard et al 1998) and were repeated in 2001. The Danish survey (Howard 2001) also asked, for some new technologies, what the expectations of Danish architects, contractors, engineers and property owners were for 2 and 5 years ahead. This data allows a pattern of growth to be established and projected beyond the year 2006. The areas in which future expectations were sought from the 1000 companies of all sizes who were sent questionnaires, were: time spent working from home, use of project web, use of Intranet and levels of investment in IT

In addition, plans for expenditure on IT applications were asked for the next two years. Some of the same questions were also asked of the same sample in 1998 and, from this data, it becomes possible to project future trends. The expected future use of applications software obviously declines with time as more firms acquire it.

**Table 1. Projections from IT surveys of the growth of particular technologies.**

| Year | 1998 | 2001 | 2003 | 2006 |
|---|---|---|---|---|
| Time working from home | | 11.5% | 14.5% | 15.1% |
| Use of project web | | 23% | 35% | 37% |
| Use of Intranet | 12% | 29% | 43% | 44% |
| Applications, expected use: | 1998 - 2000 | | 2001 - 2003 | |
| CAD | 58% | | 51% | |
| Product models | 5% | | 3% | |
| Internet | 46% | | 35% | |
| EDI / e-trade | 16% | | 15% | |

The slower rate of growth expected after 2003 could indicate the difficulty that companies often have in thinking more than two or three years ahead, and the survey obtained more optimistic results when asking which technologies would be useful at some unspecified future time.

## 3 CLASSIFICATION SYSTEMS

The Centerkontrakt project on building classification, www.byggeklassifikation.dk, has the aim of developing a better, and integrated, classification of building elements, schedules of rates and product data. Denmark currently uses a local version of the SfB system called BC/SfB. The 2001 IT Barometer survey shows that 35% of the responding companies use it and a further 34% know of it. It was developed in Sweden over 50 years ago and needs revision to relate to new forms of construction and current sources of product and price information. Companies and individuals responsible for providing this information in Denmark, as well as designers and contractors, are involved in the project.

A new system is being developed from the bottom up to suit the needs of Danish industry. The research at DTU is adding a top-down view by exploring recent developments in building information systems and building modelling which are aimed at international use. Experience from Holland, with the Lexicon multi-lingual reference library, Norway with POSC/Caesar from the process industry, Sweden with its replacement for SfB, BSAB, and the UK with Uniclass, are all reported in 'International practice and experience' (Howard 2001A)

These developments in other countries all relate to building models and libraries of building objects that are being defined in STEP, ISO 10303, or the Industry Foundation Classes. If object modelling technology is successful, and it offers a much more comprehensive use of CAD than current layering structures, the IFCs are likely to provide the modelling basis which the construction industry needs. If they are successful, and the survey did not indicate a high level of need for product models, any classification system used in Denmark needs to relate to the IFC structure. In Sweden, a study of the relationship between IFCs, BSAB and the documentation standard ISO 12006-2, has been carried out for the IT Bygg och Fastighet project (Ekholm 2000). A similar study is needed on the proposals from Denmark. Reference libraries of objects are also important, and the multi-lingual approach taken by Lexicon is necessary for countries with minority languages, such as Denmark, which wish to participate in international building projects.

## 4 THE FUTURE SCENARIOS

The methodology used to establish 5 and 10 year expectations in workshops with experts, was to identify the relevant technologies and, using any data available, discuss what would be possible over these periods. The necessary conditions for success of these were then considered in relation to general developments in IT, classification, process improvement, legislation, etc. The obstacles likely to limit the level of take up were also discussed, and a tabular scenario produced summarising the views from each workshop. Following comments by the participants, these rather technical summaries are developed into presentations using any data available and illustrating the impact of such developments on typical businesses.

These scenarios are then sent out to a representative sample of companies who are asked to answer a series of questions on their probable take-up of the new technologies within the 5 and 10 year periods. The tables that follow are summaries of the technical data from the workshops.

**Table 2. Summary of Shared project information workshop**

| *1. Technologies* | *Up to 5 yrs* | *5 years ahead* | *10 years ahead* | *Conditions* | *Barriers* |
|---|---|---|---|---|---|
| **PROJECT INFORMATION**<br>**Classification**<br>**Intelligent search** | New classification system<br>Dictionary of terms<br>Intelligent search | | Search by function for components.<br>Search on major elements | Continuous category update. Expected developments:<br>Customisation, product based<br>One solution, process based | |
| **4.1 Object technology** | GDL (Geometry Def. Language) will include more than geometry.<br>IFCs in CAD systems and some product data | | Producers sell whole elements or components.<br>IFC objects contain design intent and parametrics. | Change of design process from document to model<br>Client to demand change, as a lead user. Public authorities should lead. | |
| **4.2 INTRARET**<br>**4.3 Mobile technology** | Mobile GPS for use in logistics | Mobile phone plus computer<br>Barcode products<br>WAP is replaced | Online work instructions from integrated device.<br>http used in place of WAP. | UTMS or higher bandwidth | Possible problem with chips to suit Denmark |
| **Man-machine interface** | Click interface | Chips or barcodes in components.<br>Clients visualises standard buildings | Work instructions via animation - Bluetooth<br>Visualisation of unique buildings | Better computer screens.<br>Integration of different technologies | |
| **4.4 Documents /models** | Paper drawings | Electronic paper but still uses documents | New communication methods eg Virtual reality allows laypeople (clients) to use 3D info | Electronic paper<br>Education to work with models.<br>People need object orientation. | Personal filters for 3D.<br>Continuing education |
| **4.5 Sensor technology** | | Sensors in components for management input. | Sensors in most components<br>Management by client or by a firm hired to do this. | If managed by the owner, needs standard classification<br>If by another firm, they could use their own system | |
| **PROJECT WEB**<br>**Web project management** | Exchange of files by operator | Project webs by engineers on most new projects | Management phase supported by web tools | More openness & willingness to share info with partners.<br>Web tools should help design by architects & engineers | Benefits in use are hard to establish. |

**Table 3. Summary of Building Modelling workshop**

| *2. Technologies* | *Up to 5 years ahead* | *10 years ahead* | *Conditions* | *Barriers* |
|---|---|---|---|---|
| **BUILDING MODELS**<br>**The digital building model** | All use a 2D, object based, model.<br>Databases contain:<br>unit prices, object type and classification.<br>Producers offer 2D objects on the Internet.<br>Model integration between consultants with output still in 2D | 3D object models with databases of properties.<br>Models used directly or with simulations.<br>Producers offer 3D objects on the net with simulation properties<br>Integration by model for consultants, suppliers and clients but not craftsmen.<br>Planning tools with Artificial IntelligenceI in limited use. | New forms of partnership in the building industry. Models require a different process.<br>New roles and new partnerships, may not be a total enterprise.<br>3D models give the same benefits for partners: programs can use the data.<br>Clients recognise value of the model & demand its use.<br>AI in decision support. | Industry organisation.<br>No incentive to rationalise the process. Bidding laws limit reorganisation.<br>Clients lack understanding of models. Which can give: better quality, shorter time, reduction of risk, better management.<br>There will be cost /benefit data for whole lifecycle.<br>Clients lack IT background and awareness, but are starting to reorganise.<br>To handle models and objects, and to go from 2D to 3D, is a big step for all.<br>Big differences in partners' education and objectives.<br>National classification must work abroad.<br>Small companies need IT competence & flexible production. Problem solved by new generation. |
| **STANDARDS**<br>**International standards** | An IFC standard for object descriptions is partly complete | IFC standards depend on the market and software houses | Development and use of a modelling standard with a total classification system - national & international. | |
| **Advanced technology production** | Producers to take more part in process - planning and design. | Producers integrated into the process help implement IT. | Producers change their role to producing systems. | |
| **PRESENTAT'N**<br>**Representation technology** | More refined tools: the web, document management, project webs, multimedia.<br>VR used to sell, by demos to client. | Automatic output from the model for drawings, pictures and animation.<br>User and client more central. VR in planning and communication | Technology is developing fast in the entertainment, military and other fields.<br>There is much available international research. | |

**Table 4. Summary of Product information workshop**

| *3. Technologies* | *Up to 5 years ahead* | *5 to 10 years ahead* | *Conditions* | *Barriers* |
|---|---|---|---|---|
| **PRODUCT LIBRARIES**<br>**Object orientation** | IFCs will be implemented in XML by CAD vendors | | Changes to data structures used in building | |
| **Metadata** | BPS 130 structure plus extensions | | Demand will come from large firms | |
| | TUN metamodel of its data on wood products. | Multiple views of<br>databases FASE, IFC | Implementation of new classification system | |
| **STANDARDS**<br>**Industry Foundation Classes** | Needs experience of use in big firms - | - and transfer to smaller ones | Incentives for transfer to small firms. | |
| **EbXML/ bcXML** | AutoCAD 2002 in XML.<br>Microsoft .xp and integration with VISIO | Conversion of databases to XML | Integration of existing databases | |
| **INFO SUPPLIERS, TENDERS**<br>**Public/private** | Product info in PDF format to prevent changes to data. | Better formats | Build, Own and Operate tender | Current tendering system |
| **GENERAL**<br>**Electronic identification** | GPS in concrete elements to track their location | Bar codes imbedded chips | | |
| **International trade** | Open tendering | Global business | Tendering on value rather than lowest cost. | Competitive advantage |
| **Project Webs** | Transfer of general design information electronically.<br>Keep for full building life | Transfer at product data level | Contractor or client should define method of exchange | Need to inform client of benefits |
| **Product/process models** | Integration of project and product databases at TI | Assembly models of groups of products. | Life cycle economy | |
| **Mobile communications** | Fast Internet to mobiles | Electronic paper | | |

## Table 5 Summary of e-business workshop

| *4. Technologies* | *Up to 5 yrs* | *5 years ahead* | *10 years ahead* | *Conditions* | *Barriers* |
|---|---|---|---|---|---|
| **E-BIDDING & TENDERING**<br>**E-bidding** | Electronic tendering within a year | | Electronic tenders on all projects<br>Producers take part in design - simpler bidding with less work | Contract law<br>Frame conditions<br>Digital signatures | Digital signatures<br>Company admin<br>New roles/liabilities |
| **Value chain** | Use and support of systems in the present value chain | | New value chain, not including wholesaler or dealer. | | Producers want to keep value chain, will not offend clients. |
| **E-BUSINESS**<br>**Product service and advice** | Large suppliers offer tools and advice on products | Large suppliers/ companies put guidance on Internet | Small companies give installation advice on the net.<br>Servicing of products is often included in the package eg servicing of electrical systems | | |
| **Knowledge business** | Distance learning<br>Commercial learning communities for exchange of experience and business | | Focus changes from technology to people.<br>New technology supports this digitally | The industry should admit that it is a knowledge industry | Knowledge has a cost.<br>Too little development by companies. |
| **Mobile technology** | Better access to information In critical situations, mobiles can be used to solve problems or buy components. | | | | Successful e-business needs more data in a bid and planning for what's needed when. |
| **SUPPORT FOR BUSINESS**<br>**Cost control** | Continuous updating and pricing | Integration of storage, design & cost | System for just in time building and work on the site, eg with reporting of tasks | | Various suppliers of systems can hinder integration |
| **Teaching knowledge and e-organisation** | More project & less company oriented. Loyalty to the project results in more knowledge of it | | Big dinosaurs will use many small companies as partners since they are more innovative.<br>Middle size firms will disappear. | Holistic manage-ment.<br>Partnering creates more knowledge. | Various systems can hinder reducing skill levels and therefore hierarchies. |

## Table 6. Summary of Standard descriptions workshop

| **5. Technologies** | ***Up to 5 years*** | ***5 years ahead*** | ***10 years ahead*** | ***Conditions*** | ***Barriers*** |
|---|---|---|---|---|---|
| **STANDARD DESCRIPTIONS**<br>**Mobile devices** | Third generation alphanumeric data. | High definition small screen | Flexible large screen | | Danish building not productive or profitable |
| **Wider use of BPS type specifications** | Will be adopted by supporting firms | | | | |
| | | Bygningsdele Index | National standard Building information | Tendering more open due to changes in the tendering laws | Building industry needs restructuring |
| **COST DATABASES**<br>**Standard cost inf o structure** | Integration with V&S Byggedata, etc | | | Lifetime economics – private finance | |
| **Cost data on the Internet** | Many portals | Fewer portals | Danish integrated building database | More rules to ensure international tendering | Cultural change slow, particularly small firms |
| | Technical information on Web databases | Mass purchase of materials | | More multi-national companies in Denmark | |
| **CALCULATION**<br>(Not engineering)<br>**Attribute data in building objects** | IFC applications linked by BLIS | Integration of applications | | New forms of project collaboration | |
| **GENERAL**<br>**Integration via models** | Microsoft .net<br>VISIO – IFCs | Object databases, Project planning | Common building models | International standards | |
| **Integration tools** | Multi device<br>XML integration | Involve clients in process | Driven by user power | | Need for client to drive new processes |
| **Documents – models** | Object libraries | | | | Changing from 2D documents to 3D models is difficult. |

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These tables are summarised and therefore may be cryptic, but the information will be made more explicit and illustrated when they are tested on typical firms in the construction industry. They represent the views of about 30 experts and interrelate strongly, since the same technologies affect different applications. The implications for classification, of the technologies industry expects to use, will influence the final development of a system for Danish building for at least the next ten years.

## 5 CONCLUSIONS

At the time of writing the paper the conversion of the scenarios into an explicit form, and the industry response to these, have not been completed. The experts involved in creating the scenarios, and the results of the IT barometer survey, have been very helpful in identifying future effects of IT systems on classification. The project has also shown how difficult it is to predict technology take up more than two years ahead. The construction industry is notoriously variable in the level of its workload and plans are not often made for longer periods, certainly not by the small firms in the Danish building industry.

Some technologies, as was the case with the Internet, are likely to develop faster than anyone expected and have a much wider effect on business. Even such a now vital technology can waver in its progress, as seen with the recent decline in dotcom companies. However this new way of doing business appears to have some obvious advantages and, although the initial enthusiasm was excessive, there should be sustained growth after a temporary set back. A recent indicative graph from the Gartner Group (Ingenioren 2001) indicates a new pattern for technology development compared with idealistic S-curves. It shows a peak for e-business in 1999 followed by a dotcom decline, and suggests that a period of steady growth will follow, Fig 2. Few of the technologies specific to the construction industry are likely to suffer from similar levels of speculation.

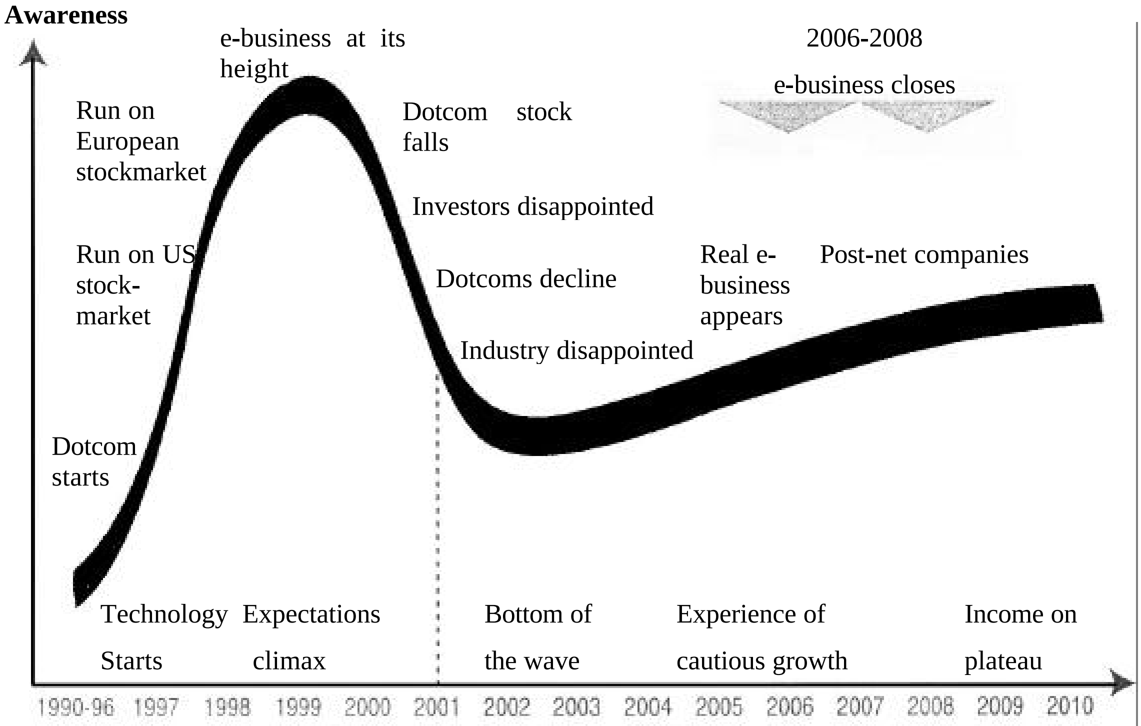


**Figure 2. The growth, decline and future consolidation of e-business. Ingenioren/Gartner group**

e-business offers a fundamentally new way of trading which many companies find hard to understand. They will take a long time to make the process changes necessary to take advantage of it. The prediction of an end to e-business in 2006-8 indicates that the first wave of such business will end but what replaces it must also be electronic. Certainly the IT barometer survey indicated significant use of e-business already, starting with retrieval of product data, and predicted a steady growth in the proportion of business carried out electronically, although it did not look beyond 2006, Fig 3.

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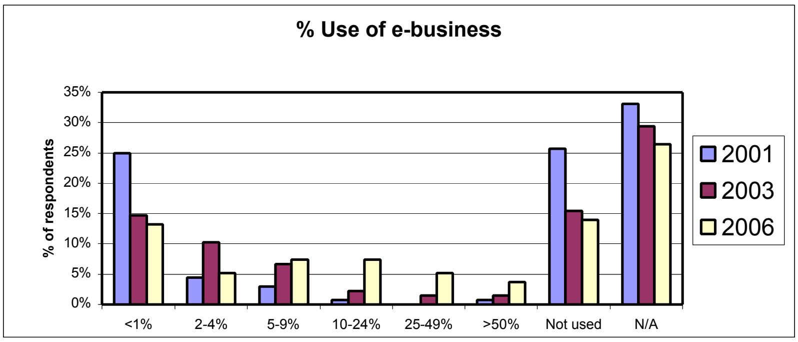


**Figure 3. Proportion of business carried out electronically 2001-3-6. IT barometer, Denmark**

Classification is fundamental to any widely used information system. Information on the World Wide Web, which is chaotically structured, is only useful through clever search engines, and this may be the solution to finding building data. However even search engines need a structure in which to report information and the results of the Centerkontrakt should be more useful for this than the various structures used currently, and could be used to integrate all the information available in Denmark.

Reliability of data is another important factor and this depends upon the reputation of the publisher and the resources available to maintain it. What is needed is an authoritative information provider, or broker, to structure collections of building data and make them available electronically. This would need to be done in conjunction with the firms that already provide access to data on building products and schedules of rates. In a small country like Denmark, it is feasible to have a single, leading information provider or broker, although competition is always stimulating. Such an information service should be seen in an international context so that information from Denmark is available in standard form in other countries, and vice versa. It must also be capable of working with future technologies to enable Danish companies to continue to play a large part in the European and international building process.

## 6 ACKNOWLEDGMENTS

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# Corrosion Mapping And Modelling

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**Summary**: **This paper describes possible approaches to mapping and modelling corrosion, namely: generalised tables and graphs, regression modelling, use of artificial neural network, and process simulation. The paper briefly presents advantages and disadvantages of each method. It does not, however, recommend any particular method.**

**The paper also describes the impacts of emerging technologies such as geographic information systems and the Internet that have allowed more efficient modelling, and information delivery of corrosion maps and other related information. It also describes a WWW-enabled corrosion mapping system that has resulted from CSIRO corrosion mapping and modelling work.**

**Keywords. Corrosion maps, geographic information systems, World Wide Web applications.**

## 1 INTRODUCTION -CORROSION MAPPING

Corrosion maps are traditionally constructed using two basic methods. Both methods are based on field observations from a set of sites distributed within the study area, and located in such a way that they are representative of the variability of the corrosion rates within the study area.

If enough data points are available, the most straightforward and accurate approach is to construct a surface model of the corrosion rate (e.g. triangulated irregular network, grid, contour lines). In this approach, arranging the sites in some sort of grid pattern is desirable. This approach was used in producing several corrosion maps for Melbourne (see Fig. 1), Newcastle, The Hunter Valley and South Australia. Unfortunately, this approach is very costly both in terms of financial cost and time. This cost constraint is a significant drawback in the production of any wide-area corrosion maps.

If there are not enough data points to adequately cover the spatial extent of the study area, an alternative approach is to construct a mathematical model, either based on statistical methods, artificial intelligence techniques or process simulation, or combinations thereof. Selecting the set of input variables to the corrosion model is difficult and could vary from one case to another. For example, estimating corrosion using only distance from the coast, implicitly assumes that air pollution is not relevant (see Fig. 2). This implicit assumption has resulted in the underestimation of corrosion in industrial areas (see Fig. 3). Hence, parametric models tend to vary in form from one area to another.

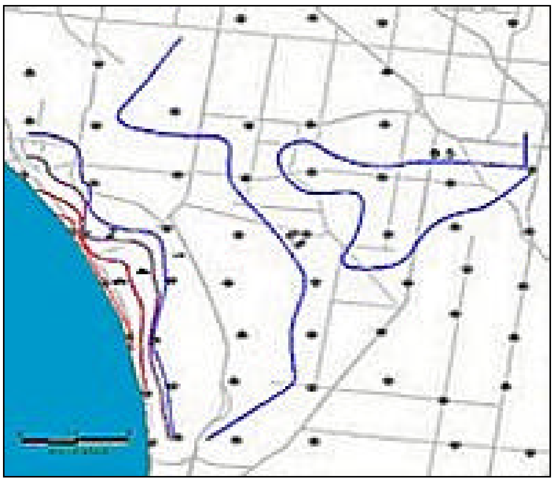

**Figure 1. Observed corrosion of galvanised steel in south-east Melbourne, 1983-85 (in µm per year).**

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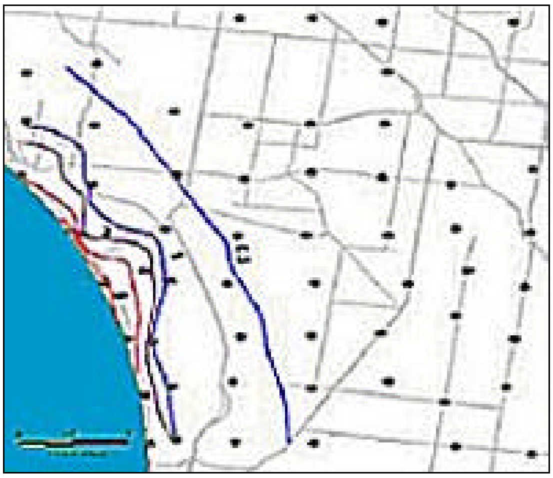


**Figure 2. Estimated corrosion of galvanised steel based on distance from the coast (in µm per year).**

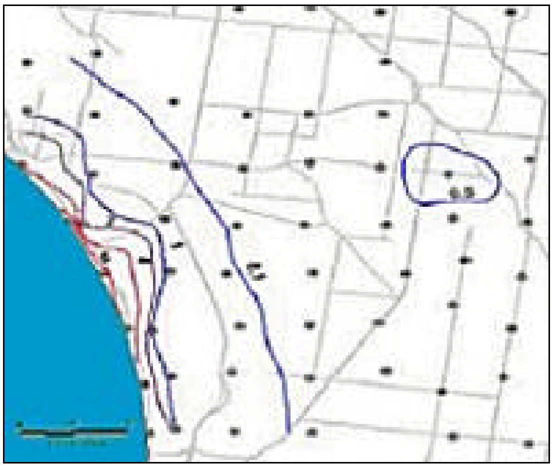


**Figure 3. Estimated corrosion of galvanised steel based on distance from the coast and $SO_2$ level.**

## 2 GIS–BASED CORROSION MODELLING

The emergence of geographic information systems (GIS) makes it possible to construct corrosion models quicker and cheaper. A GIS allows:

- Various data sets from different sources to be integrated into one unified framework.
- An incremental and interactive (possibly visual) process of building the corrosion model.

A corrosion model based on both statistical and process simulation was developed at CSIRO Building, Construction and Engineering. This model was based on the following simple assumptions:

- Corrosion is influenced by two basic factors – salinity and moisture (time of wetness).
- If industrial pollution is neglected, then distance from the coast, topography, prevailing winds and the type of coast (surf or bay) will influence salinity levels.
- Moisture is basically influenced by climatic parameters such as relative humidity and temperature.

The 'time of wetness' (TOW), or the time a metal surface is assumed to be wet, is computed from relative humidity and temperature records in 136 Bureau of Meteorology sites using a method suggested by the International Standards Organization. An Australia-wide surface model of TOW is constructed from the 136 values using a geo-statistical technique called 'Kriging'. Figure 4 illustrates the resulting surface model for the TOW.

The corrosion rate in each of the 14,700+ localities (towns or suburbs) in Australia is estimated by:

- Representing each locality as a point defined by a pair of coordinates (longitude, latitude).
- TOW at each locality is estimated from the surface model shown in Fig. 4.
- The amount of salt transported to each locality from the coast is simulated (see Fig. 5).

- A surface model of corrosion rate is derived from the corrosion estimates in the 14,700+ localities (shown in Fig. 6).

## 3 CSIRO WEB-BASED CORROSION INFORMATION SYSTEM

The rapid acceptance and increased accessibility of the Internet paved the way to a new method of delivering information. The CSIRO corrosion information system is Web-enabled, as expected of most information systems today. The corrosion information system is a component of a wider CSIRO initiative, which is the Build Information Exchange (BIEX) Internet portal (see Fig. 7).

The resulting corrosion maps and model are delivered through the Internet in a cost-effective manner (shown in Figs 8 and 9). It is also possible to value-add to these models and maps by coupling them with service life estimation and material selection applications (shown in Figs 10 and 11).

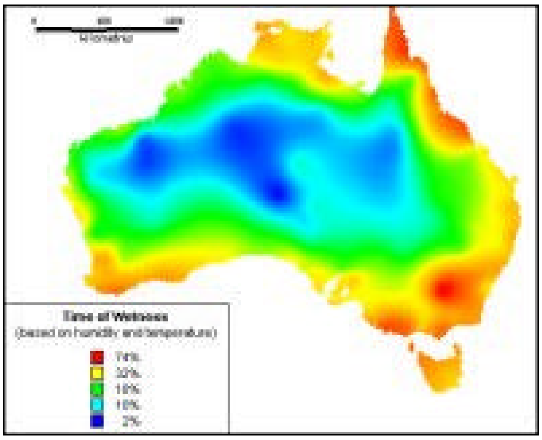



**Figure 4. Estimated TOW in Australia based on meteorological data.**

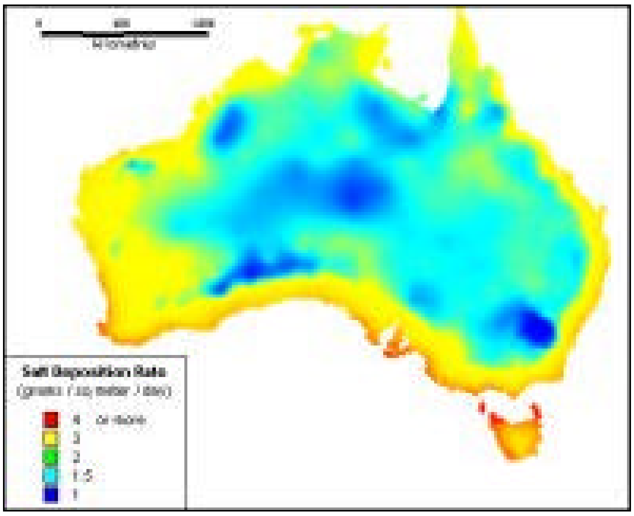



**Figure 5. Simulated salt deposition Australia-wide using meteorological and coastal data.**

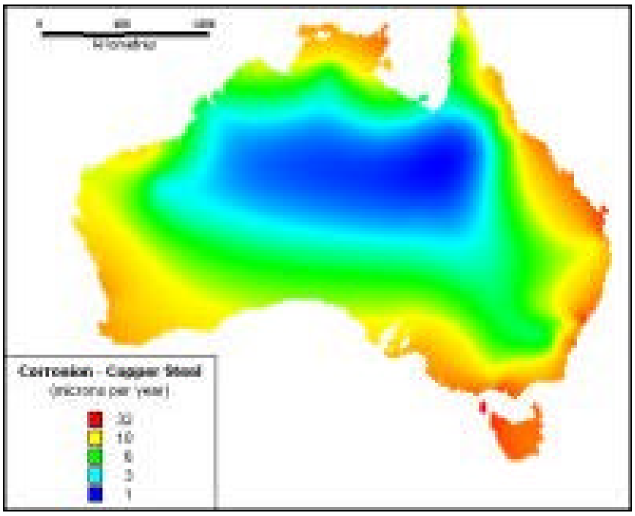



**Figure 6. Corrosion map for copper steel derived using geo-statistical modelling.**


**Figure 7. CSIRO Build Information Exchange Internet portal**

(http://www.dbce.csiro.au/biex/main.cfm)

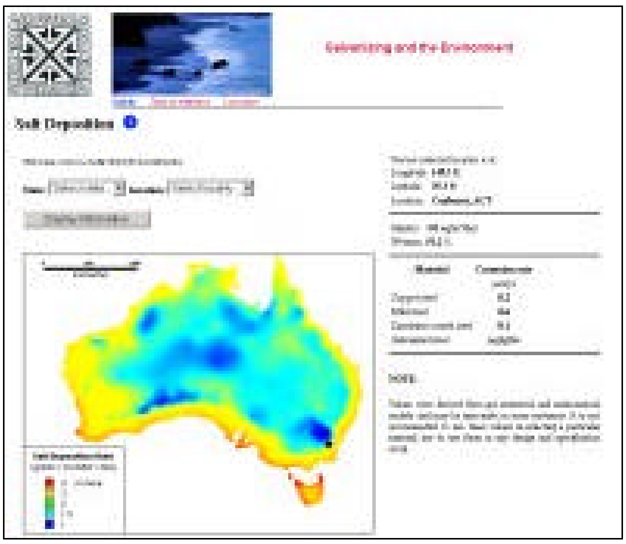



**Figure 8. The corrosion mapping system incorporated with the Industrial Galvanisers Corporation Website.**

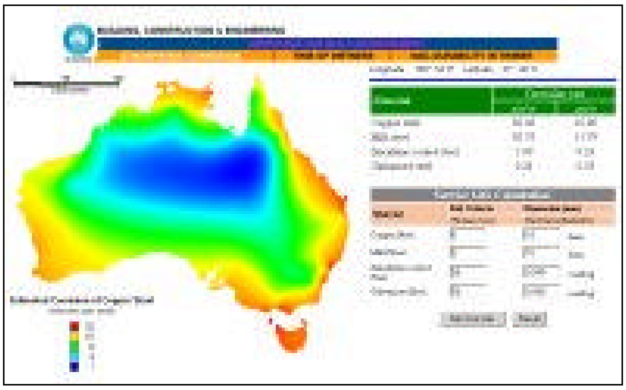
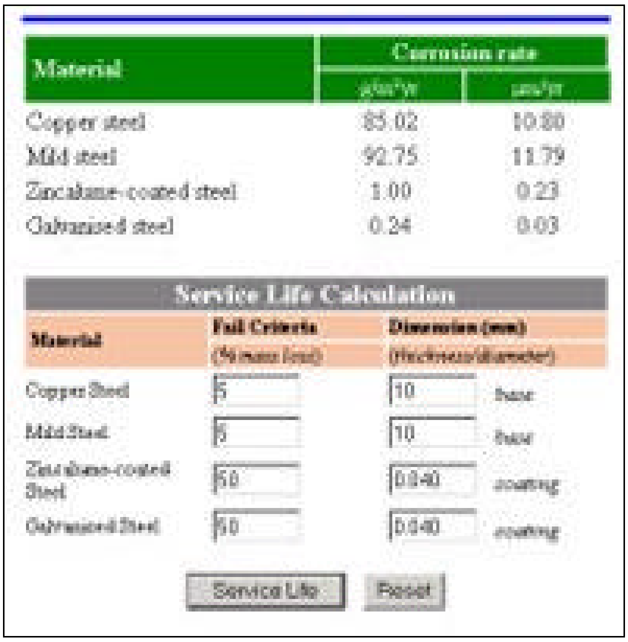


**Figure 9. The corrosion mapping system embedded within the BIEX portal.**



| Material | Corrosion rate | |
|---|---|---|
| | g/m²/yr | µm/yr |
| Copper steel | 85.02 | 10.80 |
| Mild steel | 92.75 | 11.79 |
| Zincalume-coated steel | 1.00 | 0.23 |
| Galvanised steel | 0.24 | 0.03 |

**Service Life Calculation**

| Material | Fail Criteria (% mass loss) | Dimension (mm) (thickness/diameter) | |
|---|---|---|---|
| Copper Steel | 5 | 10 | base |
| Mild Steel | 5 | 10 | base |
| Zincalume-coated Steel | 50 | 0.040 | coating |
| Galvanised Steel | 50 | 0.040 | coating |



**Figure 10. Dialog box allowing user to define the failure criteria.**

| Material | Corrosion rate | | Service Life |
|---|---|---|---|
| | g/m²/yr | µm/yr | (years) |
| Copper steel | 85.02 | 10.80 | 23 |
| Mild steel | 92.75 | 11.79 | 21 |
| Zincalume-coated steel | 1.00 | 0.23 | 88 |
| Galvanised steel | 0.24 | 0.03 | 381 |



**Figure 11. Estimated service life for selected materials from user-defined failure criteria.**

## 4 OTHER WEB–BASED CORROSION INFORMATION

There are several corrosion-oriented Internet portals. One of them is Corrosion Source, a site that is essentially a marketplace where one could buy software tools, among other corrosion-related products and services (http://www.corrosionsource.com/).

Another example is an Java-based corrosion rate and service life calculator, jointly developed by the International Lead Zinc Research Organization and Cominco, which allows the user to estimate the corrosion rate of zinc by nominating parameters on rainfall, salt deposition, sulfur dioxide levels, relative humidity, temperature and sheltering condition (see Fig. 12 and 13). The site is based on Java technology and is hosted at http://www.fortjava.com/zclp/index.html.

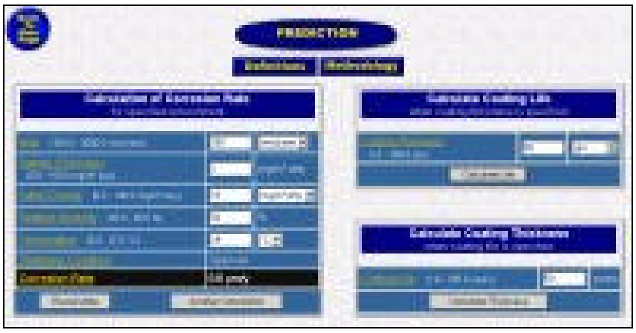



**Figure 12. Corrosion rate and service life calculator for zinc coating.**

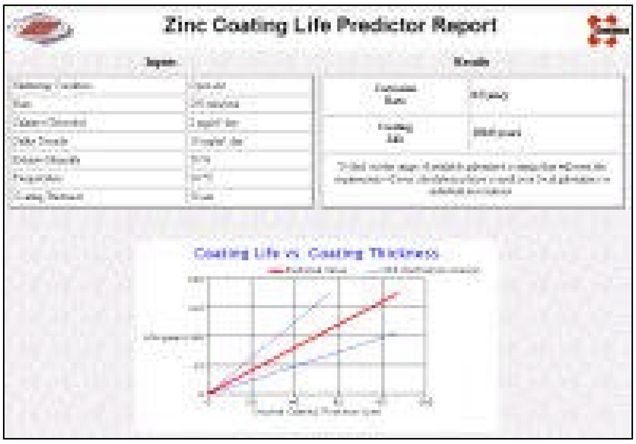



**Figure 13. Zinc coating life predictor report.**

The third example is the Corrosion Doctors Resource Web portal, which contains several predefined (i.e. static) low-resolution maps for China, India, Japan, New Zealand (see Fig. 14), North America and South Africa. The site is hosted at http://www.corrosion-doctors.org/.

## 5 CORROSION PREDICTION MODELS

### 5.1 Generalised tables and graphs

Atmospheric corrosion rates of metals can significantly vary depending on the specific environmental conditions. The most commonly used method for corrosion life estimation of galvanised steels has been the use of generalised values for the different types of predefined environmental conditions. This method uses a generalised value in the form of tables and graphs to represent the corrosion rates in each typical environment. An example of this method is the environmental corrosion classification system that was developed by the International Standard Organization (ISO 1992).

The method provides a simplified estimation of corrosion rates. Unfortunately, it is only able to provide rough and non-specific estimates. Hence, this non-specific approach is no longer adequate to meet the demands of the marketplace. For instance, present designers and specifiers of galvanised steel increasingly ask for information on performance certainty. The ‘coating guarantee program’ of the Industrial Galvanisers Corporation (IGC) is a typical example. IGC manages its own coating guarantee projects, and provides design professionals with real-time corrosion data. This is becoming more important as demand intensifies for reliable, long-term durability of steel construction products, and designers become more accountable for material durability performance.

### 5.2 Geographic mapping method

In this method, corrosion rates of materials in a geographic area are based on field measurements in a grid of sites (King 1993; Shaw 1976). It recognises the difficulty of using general corrosion rate predictions and attempts to estimate product service life directly from field data. Thus, it is the most reliable method for the estimation of product life. However, its usefulness is limited to the areas where such mapping is available, and there are relatively few regions of the world that have been suitably mapped for this purpose.

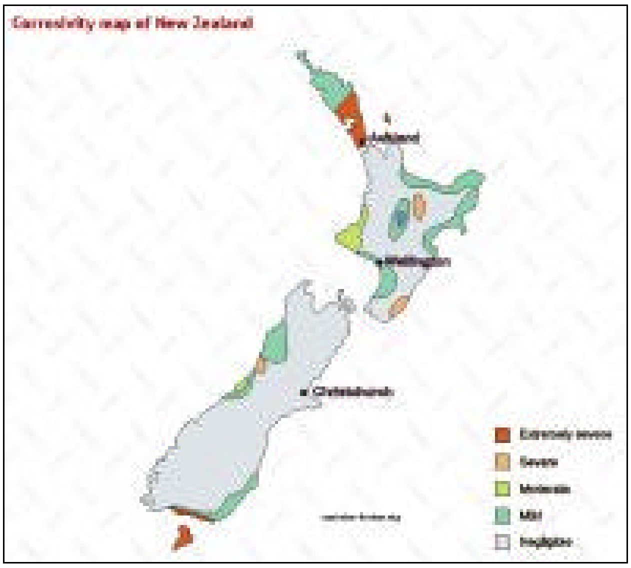


**Figure 14. New Zealand corrosion map at the Corrosion Doctors Website.**

The geographic mapping approach is fairly straightforward. Once, the corrosion map is complete, using it is as simple as selecting a point on the map. Its main difficulty is in the logistics involved in setting up a grided network of field sites, maintaining them, and analysing and assembling the results. This process is both time consuming and expensive.

### 5.3 Regression models

Regression models are derived from mathematical functions that are empirically formulated based on the statistical analysis of historical data with respect to the relevant factors. Numerous studies have indicated that corrosion rates are strongly affected by certain factors, such as TOW, sulfur dioxide and chloride concentrations in the air. Hence, it is not surprising that most regression formulae use some if not all these variables. The following are some examples of such corrosion rate formulae.

C = a ln (RH) + ß ln (T) + ? ln (Salt) (Trinidad & Cole 2000) (1)

C = a ln (RH) + ß ln (T) + ? (DistanceFromCoast)$^{-1}$ (Trinidad & Cole 2000) (2)

] C = a ($SO_2$) + ß (Chloride) + ? (Wetness) + d (Knotkova *et al.* 1995) (3)

C = [a ($SO_2$) + ß (Chloride) + ?] Temp (Kucera *et al.* 1982) (4)

Performing regression analysis is easy when the form of the regression function is known. Once the input data is assembled, the analyst only needs to feed this data to regression software. Regression function is a standard feature on any statistical software or even part of everyday office software (e.g. spreadsheet).

One of the problems with this approach is that the applicability of the resulting regression equation is somehow limited by the coverage, variability and completeness of the input data. For example, Eq. (1) is suited to South East Asian regions with no extensive mountain ranges. Hence, this equation may work well in Thailand or in Mindanao Island (Philippines), but would perform poorly in Sumatra (Indonesia) and Luzon (Philippines).

The main difficulty in using the regression method is determining the form of the regression function. Consider the example of generating a regression model that estimates the corrosion rate (C) given relative humidity (RH), temperature (T) and coastal distance (D). The first step the analyst needs to take in this situation is to assume the form of the regression equation. For instance, an analyst must first decide that the function will be of the form:

$$C = \alpha\,(R) + \beta\,(T) + \gamma\,(D)^{0.5} + \kappa$$

Only then can the analyst feed the raw data into the regression analysis module, which is typically part of most statistical software products. The difficult lies in determining the form of the function and identifying the set of variables that are best suited. It is possible that for the problem at hand, the best regression equation is:

$$C = R\,(D)^{\beta} \rightarrow \text{which reduces to} \rightarrow \log(C) = \log(R) + \beta \log(D)$$

### 5.4 Artificial neural networks

The difficulty of determining the form of the regression function is avoided by using the neural network approach. A neural network only needs to be given raw data related to the problem. It sorts through this information and produces an understanding of the factors by defining the function, $f$, such that:

$$C = f\,(R, T, D)$$

Artificial neural networks are the result of academic investigations that involve using mathematical formulations to model nervous system operations. The resulting techniques are being successfully applied in a variety of applications. A neural network can be used to learn patterns and relationships in data. Traditionally a programmer or analyst specifically 'codes' every facet of the problem in order for the computer to 'understand' the situation. Neural networks do not require the explicit coding of the problem.

Unfortunately, this approach also suffers from several drawbacks. First, it is a 'black box' approach. It is difficult for the analyst to understand the formulation process since this is done within the neural network software. Second, it also suffers from the deficiency of regression modelling such as limitation of the applicability to particular environment, and the usual data availability problem. Third, selecting a suitable neural network software product can be daunting to corrosion professionals. The Google Internet search engine has an indexed holding of around 50 commercially available neural network software products (www.google.com).

An example of a neural network application in corrosion modelling is the ILZRO-Cominco Corrosion Rate Calculator, which was based on a neural network-based model developed by Zhang (2001).

### 5.5 Holistic approach

To avoid having to measure corrosion itself, Cole *et al.* (1999) proposed the 'holistic' approach. It proposed to simulate the corrosion process itself, from the salt transport mechanism, particle deposition, film or surface formation, and other processes. At its purest form, the holistic approach is extremely difficult if not impossible to achieve.

First, it is unlikely to have a complete understanding of the corrosion process. It may be possible to simulate some part of the process under some known set of conditions. However, it may not be possible to simulate the entire process under all sets of possible conditions. Second, even if one can completely understand and explain the entire corrosion process, it is computationally impossible to build such a simulation system.

A more pragmatic approach is to simulate some of the corrosion process components and use other methods in other components. This is the approach taken in the construction of the CSIRO on-line corrosion map. Table 1 illustrates the methods that were used to model components of the corrosion process.

## 6 CONCLUSIONS

In conclusion, it should be noted that among the available modelling approaches, the geographic mapping method produces the most accurate estimates and it is easy to use. However, its usefulness is limited to the areas where such mapping is available, and there are relatively few regions of the world that have been suitably mapped for this purpose. The generalised method, although simple, is no longer adequate to meet the current demands of today's marketplace.

**Table 1. An example of table layout**

| *Component* | *Method used* |
|---|---|
| Time of wetness | Generalised tables |
| Salt production | Generalised tables |
| Salt transport and deposition | Process simulation |
| Corrosion rate of copper steel | Process simulation |
| Corrosion rate of other materials | Regression model |
| Corrosion map production | Geo-statistical tessellations |

The use of regression and neural network models shared the same advantages and disadvantages. Both methods tend to produce more accurate estimates than the generalised approach and have a slightly wider applicability than the geographic mapping approach. Regression models seem a better alternative when a small number of variables are used to estimate corrosion. On the other hand, the use of neural networks is more advantageous if larger numbers of variables are used in estimating corrosion.

The holistic approach, although theoretically sound, cannot be implemented with currently available knowledge and technology. The more pragmatic hybrid approach does not give accurate results when compared to the geographic mapping approach. It may be more widely applicable than both regression and neural net modelling, but it does not guarantee to produce a better estimate.

It is evident that recent developments in information technology, especially in the area of geographic information systems, artificial intelligence, Internet and interoperable database systems, have improved the corrosion mapping and modelling process.

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## 7 ACKNOWLEDGMENTS

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# A Web-Based Management Information System For Multi-Party In Construction Projects

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**Summary**: **This study aims to develop a system for the participants in construction projects to manage the outward information based on Web technology. In this paper, the characteristics of the outward information management in construction projects in China are summarized based on literature survey and field investigation, and a Web-based approach to developing the system is proposed. Then the prototype system that we have developed by using the latest information technologies including XML (eXtensible Markup Language) is examined and its applicability is discussed. The system can accommodate efficient collaborative work among the participants in construction projects and facilitates each participant to grasp the related information effectively. In addiction, the system facilitates the participants to make use of their own management information system together with this system.**

**Keywords. Construction projects, Management information system, Collaboration, Web-based**

## 1 INTRODUCTION

Every moderate to a large construction project involves participants such as owner, designer, engineer, contractor, subcontractor, supplier etc. Due to the complexity of construction process, information management plays a very important role in the whole construction process.

According to the scope of information, the information management in a construction project can be divided into two categories, i.e. the internal information management and outward information management. The former denotes that within each participant, for example, the information management within a contractor; while the latter denotes that across at least two participants to collaborate their activities. In the past ten years, the way of the former has changed a great deal due to the use of information technology, while that of the latter remains the old fashion, i.e. the information is submitted in paper form and checked by signing manually.

Major shortcomings can be identified for the old fashion of outward information management. The primary one is that it blocks the flow of digital information among multi-party. Since the information in the computer in one participant has to be printed out on paper in order to be submitted to another one, then the paper form information has to be input into computer in the other participant to manage it by using a computer. As a result, the movement of information is time consuming and expensive. The remarkable time and cost needed for printing out the information, sending the paper form information by mail or in person and inputting it into computer explain why it is. Obviously, the shortcoming can be overcome by utilizing the Web technology that prevails in recent years.

Researches on collaborative systems for Architecture/Engineering/Construction (AEC) industry have been carried out for several years. Teicholz and Fischer (1994) proposed the concept of Computer Integrated Construction (CIC) to integrate the project participants in design and construction based on object-oriented model. Anumba (1996) pointed out the need for tools that can support collaborative environment for construction management. COMMIT project (Rezqui 1998) clarified for the first time the important issues regarding the information management in collaborative environments, such as ownership, versioning etc. These studies partly solved the basic problems for the collaboration of participants in construction, but they are not applicable to the practice as a whole. Ma and Itoh (1999) conducted a study on road lifecycle information management systems and developed a prototype collaborative system on Internet. However, it was not flexible to adapt to new specifications on information management.

This study aims to develop a system for the multi-party in construction project to carry out the outward information management based on Web technology. In this paper, the characteristics of the outward information management in construction projects in China are summarized based on literature survey and field investigation, and a Web-based approach to developing the system for outward information management of construction projects is proposed. Then the prototype system that we have developed based on the approach by using the latest information technologies including XML is examined and its applicability is discussed.

## 2 CHARACTERISTICS OF OUTWARD INFORMATION MANAGEMENT

Since late 1980s, a new delivery mechanism has been introduced in construction industry in China to cope with the market economy. In the mechanism, owner, designer, contractor and engineer constitute the major participants for a construction project and they all collaborate according to the contracts that have been signed between them and the specifications issued by the government (BMMC 1998, BMMC & BMPC 2000). Imaginably, information movement is indispensable in the collaboration.

The outward information, which is conveyed among the multi-party for the collaboration, includes that regarding scheduling, cost control, quality assurance and contract management. Each type of information has its own specific forms. For example, the information regarding scheduling includes the report form of construction start, that of total schedule, that of monthly schedule, that of monthly statements, notice from engineers etc. Each form should be prepared and submitted by a certain participant, checked and/or determined by others. For example, the report form of monthly schedule should be prepared and submitted by the contractor, determined by the engineer. In addition, some of the forms have to be submitted along with the related documentation. For instance, the report form of monthly schedule has to be submitted along with the plan for the schedule, quantity survey, and list of equipment.

Normally, the information that is conveyed in the construction process is not only necessary but also meaningful for the related participants in different ways. For example, by submitting the monthly schedule, the contractor promises the progress of the project; the engineer compares the monthly statements with the corresponding schedule to check if the project goes as scheduled; and the owner prepares the upcoming payment according to the schedule. In addition, each participant may examine the statistic of certain information items such as payment amount.

It deserves to note that all the information forms are specified in the government specifications and subject to change every several years. Besides, depending on the characteristics of the construction project, some forms may need to be created for the project.

## 3 APPROACH

At least three alternative ways in Internet are available to be used for conveying information among multi-party. They are email, FTP (File Transmission Protocol) and Web-based file folder systems (Buzzsaw.com Inc. 2001). Email is used nowadays as a common way to send message and attached files. But it is difficult to check whether the message and attachment files have arrived at the destination. FTP is used to send files to a specified FTP server, which can be made visible for multi-party. While these two ways provide only the basic functions for conveying information, Web-based file folder systems provide further capability. It allows to establish a workspace in the form of folder tree of files, and to assign a super user to maintain the structure of folder tree. The super user can define normal users and their right to access the files under folder tree such as browse, upload and download. In this way, the defined users can use the specified workspace to convey information.

However, the above-mentioned ways are insufficient for outward information management in construction in several aspects. First of all, the minimum object that is handled is a piece of message or a file, but the practice in construction needs to handle the contents of the message and/or files, i.e. information items. Secondly, the user has to be aware of the location for storing the message and files when he wants to store the information hierarchically, while it is desirable that the information is stored in a hierarchical structure automatically.

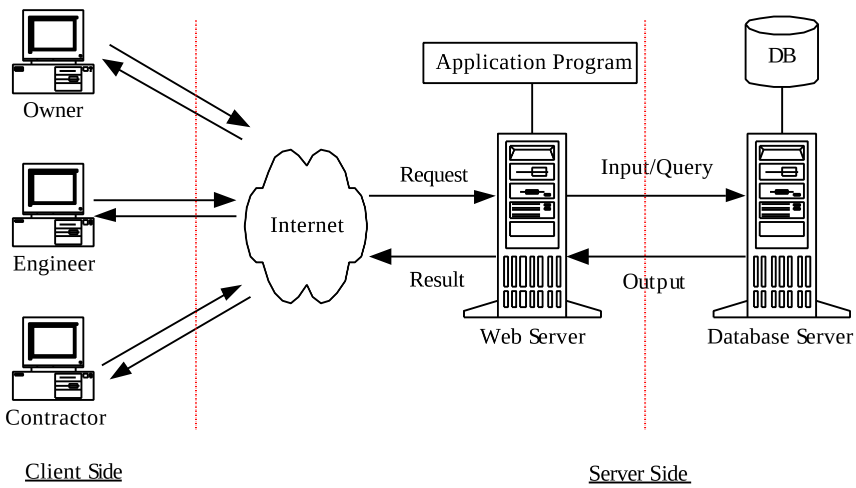


**Figure 1. System architecture**

In order to solve these problems, the approach that we have taken in this study is to develop a Web-based system for the outward information management of construction. A three-tier architecture is adopted, as shown in Figure 1. It is assumed that the owner of the construction project or the authorized ISP (Internet Service Provider) maintains a Web server and a database server. The application program that we develop in this study is installed in the Web server and the DBMS (Database Management Systems) is installed in the database server. The client machines used by the multi-party users are only required to install a Web browser and connect to Internet.

A hybrid database is used in the application program for managing the form data in the outward information management, as shown in Figure 2. The database is divided into three parts, i.e. the relational database part, the XML document part, and the attachment file part. Here, the relational database part is used to store the common information of the forms, i.e. the header; XML document part is used to store the varied part of form data, i.e. the contents; and the attachment file part is used to store the attachment files, i.e. the appendix. As it is known, XML is a markup language that uses tags to store structured information items in a file (Harold 2000, Sturm 2000). Each XML file has a corresponding schema file that is used to define the tags, and at least one corresponding style file that is used to rectify the display style of the XML file. In this sense, the schema file and the style file together can be regarded as the template of the forms.

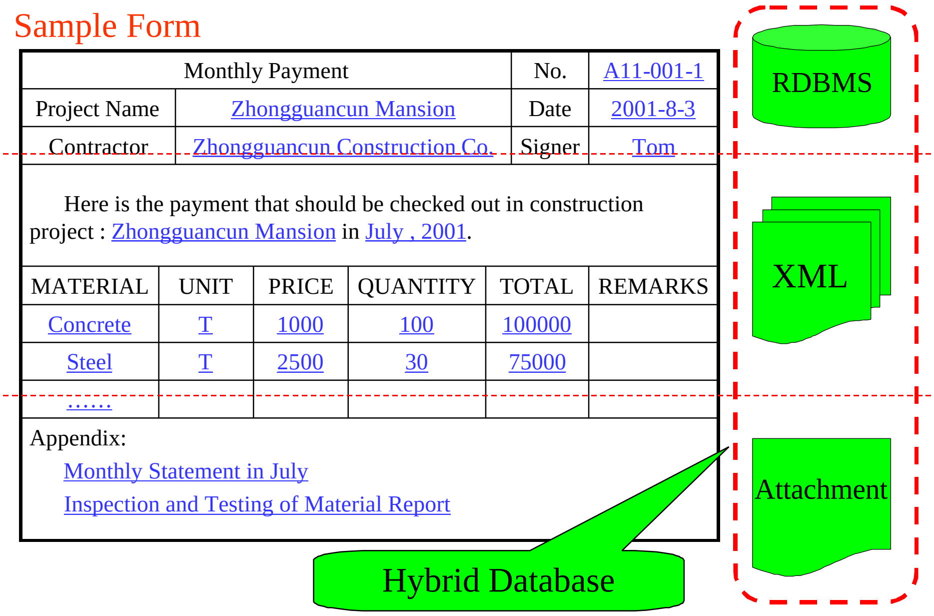


**Figure 2. Concept of the hybrid database**

By using XML, the application program developed in this study is made very flexible. That is, in addition that the minimum object to be handled by the application program becomes information items, the forms can be changed without updating the code of the application program. The only thing that we have to do is to create the templates of the forms and then import them into the application program after removing the outdated templates of forms. In addition, it deserves to explain that the relational database is used because its adequacy in handling the relational data and its high speed to access the data compared to accessing the XML files.

## 4 A PROTOTYPE SYSTEM: EPIMS

### 4.1 Development environment

A prototype system has been developed in this study. Microsoft NT 4.0 is adopted as OS, and Microsoft SQL Server 7.0, which supports concurrent data access, is adopted as the DBMS. Microsoft Visual Interdev 6.0 is used as the development environment of the application program. The application program is called EPIMS (Electronic Project Information Management System). It is coded mainly by using ASP (Active Server Pages) and Microsoft VB Scripts. Microsoft XML ver. 2.0 is used for analyzing XML files in EPIMS.

### 4.2 Major functions

In EPIMS, the object of information for submitting, checking and determining is a document that may include the form data, attachment files, and links to other submitted related documents. Except the functions for project setting, most of the functions in EPIMS are provided around the documents.

The functions are divided into six categories and each category contains several menu items, as shown in Table 1.

**Table 1. Major functions of EPIMS in the system**

| *No.* | *Category* | *Is used for…* | *Menu items included* |
|---|---|---|---|
| 1 | Project setting | conducting settings that is particular for the current project, including the multi-party, the user in-charge of each participant, the template of documents, the access rights of multi-party for documents etc. | Set participant; Set participant super user; Select from system; Build new template; Set access right for participant; Set project preferences. |
| 2 | Browse and Handle | browsing the related documents in different viewpoint such as the type of document, and checking and/or determining the related documents | New related documents; By type; By handling type; By sub-mitter; By submission date. |
| 3 | Submit | submitting documents by filling in the form or by specifying an existing XML file | By filling in; By using existing file. |
| 4 | Query and Present | querying on both attributes such as submission data and contents such as a certain information item of the documents, and providing graphical presentation on any item upon specified conditions | By attributes; By contents; Comprehensive query; Graphical presentation. |
| 5 | Notify | sending non-professional notice and browsing this kind of notice | Browse notice; Send notice. |
| 6 | Others | browsing the information of the project and for the user in-charge to set the user and the access right of documents for each user in his own company | Project information; Company/user information; Set user; Set access rights. |

## 4.3 Typical use cases

### 4.3.1 Set the access right for users

The setting is done against each document type. Setting the access right for users means to specify who have the right to handle, i.e. to submit, check, determine and/or browse the document type. It is done through two steps in EPIMS. The first step is to specify which participant in what way can access the document type. For example, the report form of monthly schedule should be prepared and submitted by the contractor, determined by the engineer. This is done by executing the menu "Project setting>Set access right for participant", as shown in Figure 3. In addition, some document types have to be checked by the in-charge user before being handled by other participants. The second step is used to solve this problem. It is used to specify who can submit and/or check the document type in the specified participant. This is done by executing the menu "Others>Set access right", as shown in Figure 4.

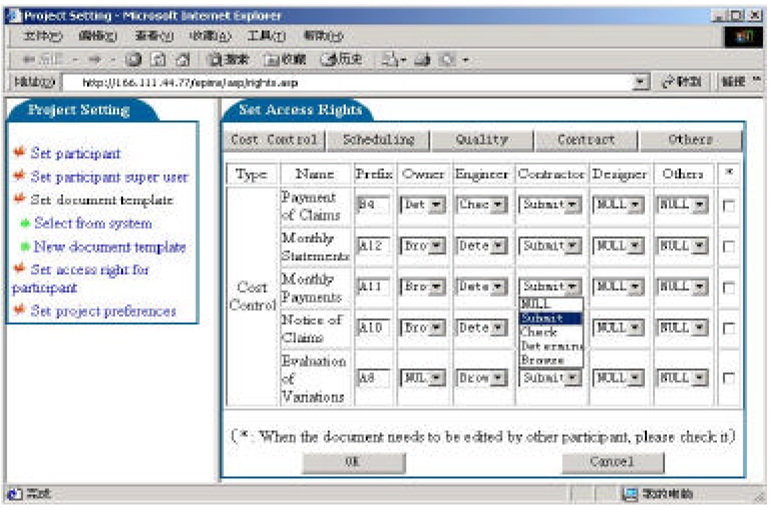


**Figure 3. Set access right of documents for multi-party**

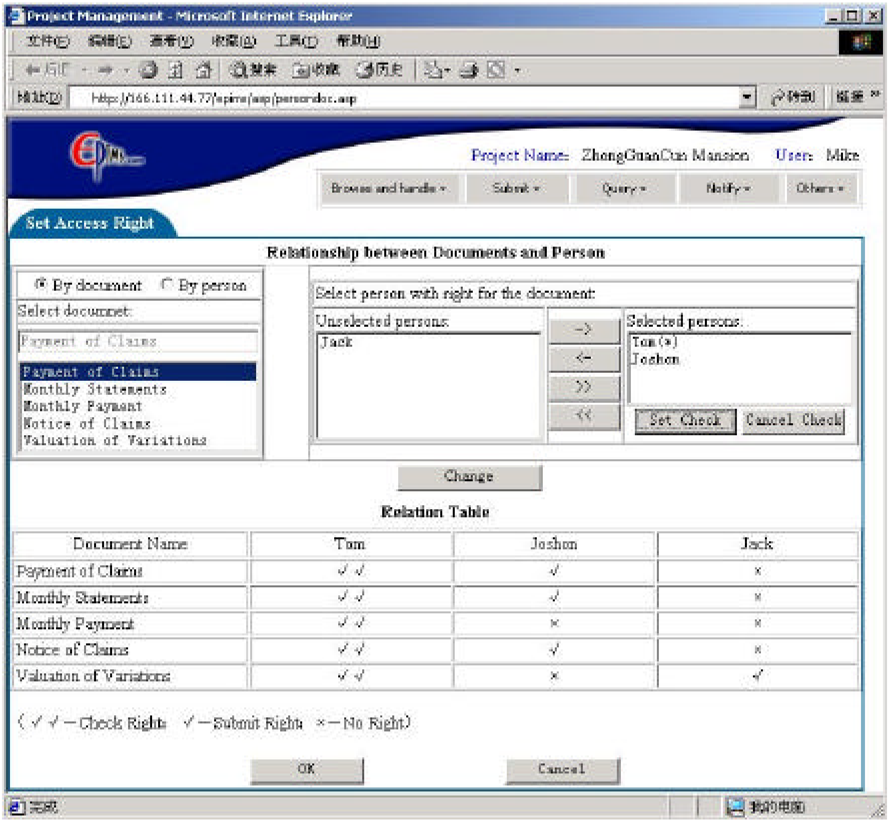


**Figure 4. Set the submission and check rights of documents**

### 4.3.2 Submit, check and determine a document

By executing the menu "Submit>By filling in", the interface as shown in Figure 5 appears for specifying the details. Here, the user can select the document type and document name and input the title of document, edit the document No., and select the attachment files and related documents. Then he needs to fill in the form with contents. Since each form is displayed by analyzing the corresponding schema file, only the title of information item is displayed, which may not be so clear to the users. In order to make up for this, a button "Sample" is provided to help users understand the information items in the form by showing a filled original form.

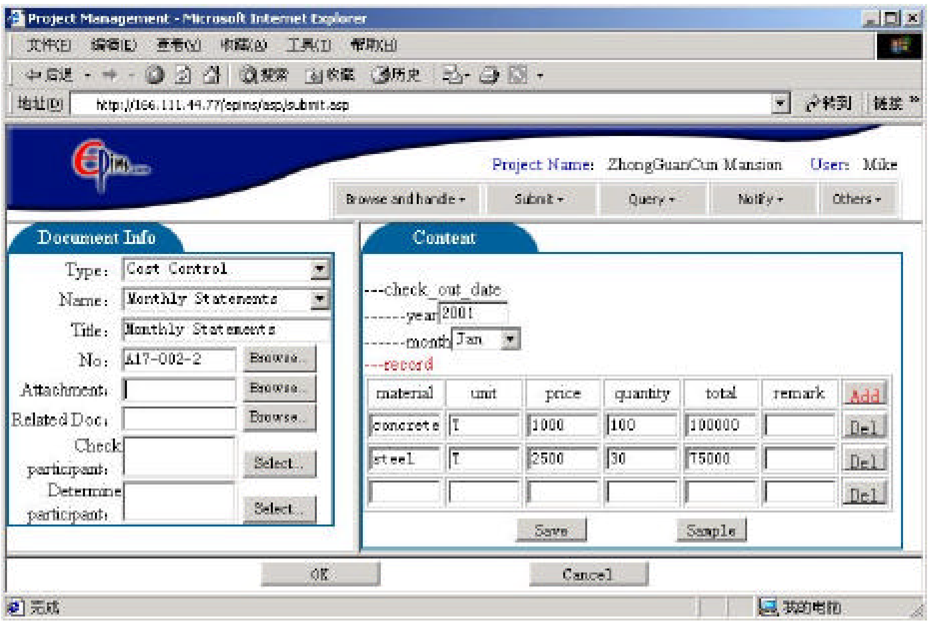


**Figure 5. Submit by filling in the form**

After this moment, if the related user from another participant enter EPIMS, he will find in the list of "to be checked" that a document is pending for him to check or determine. The interface is shown in Figure 6. Here, he can browse the document and he can also retrieve the relative document to refer, then he finishes this process by filling in the text box and pressing the buttons "agree" or "deny". He can also modify the document and then to agree. In this case, a new document is created to replace the old one. Since the workflow, which defines the sequence of document handling and is set in the "project setting", is controlled by the system automatically so that multi-party in construction projects can collaborate efficiently.

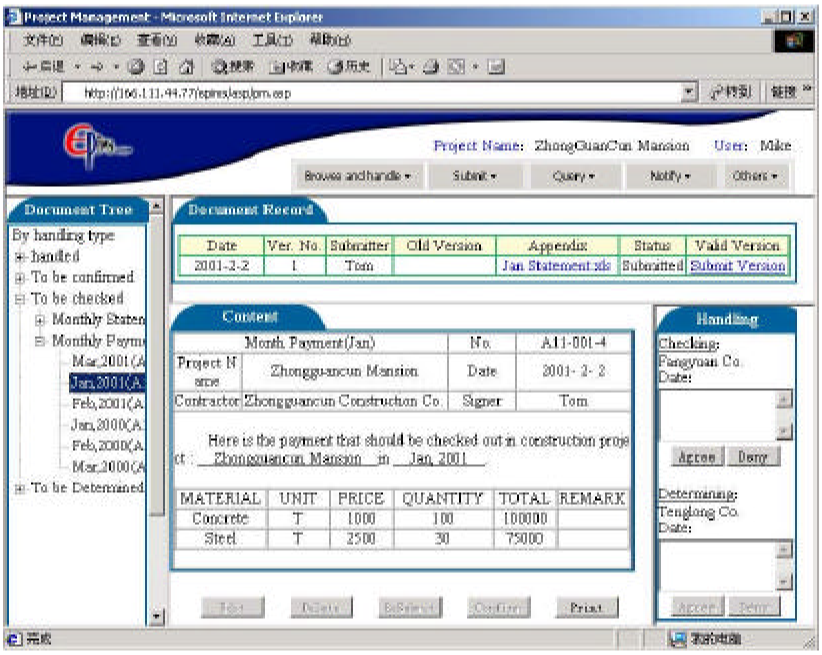


**Figure 6. Main interface for browsing and handling information**

4.3.3 Obtain graphical presentation on the query result of a document type

When a user wants to grasp the statistic of a certain information item, he needs to execute the menu "Query>Graphical presentation", as shown in Figure 7. He can specify the query condition to confine the scope of information to that of concern and then to specify the variable and value items to present the result graphically, i.e. in the form of histogram, line, pie graph etc.

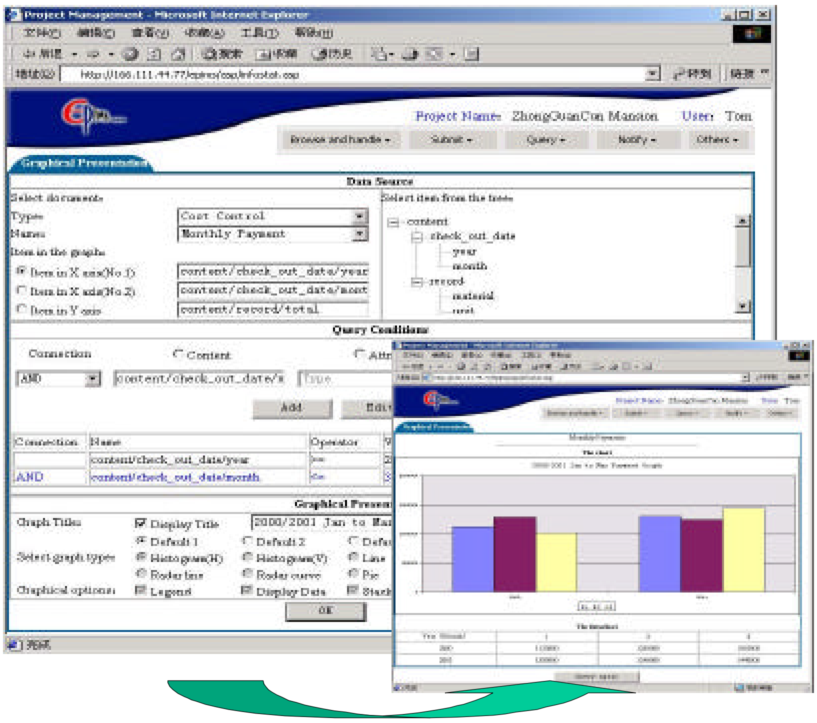


**Fig 7. Graphical representation of the query result**

Here, he can specify the condition as a combination of "attribute" and "content". It deserves to notice that the schema file are analyzed by the program and the tree of information items appears in the interface so that any information item can be selected when specifying the query conditions.

## 5 DISCUSSION

Although this approach seems to be able to solve the problem of outward information management in the aspects of speed, efficiency and accuracy in construction projects, mainly two questions have to be answered about the applicability of the system.

One is whether this approach can adapt to the current practice. The focus lies in if the check made by a user in the system can replace his signature in traditional paper form, while there is not any law about electronic signature in China. The answer is yes. This problem is solved in two levels in the approach. The first level concerns the construction process, i.e. each participant in the project reaches an agreement on the electronic check. Then they can use the system at least for the construction process to gain the merits mentioned above. This level is comparable to the early practice in EDI (Electronic Data Interchange) (Mizuda & Mitsuhashi 1997). The second level is to cope with the requirements on documentation by current specifications. The method is to print out the relative reports and gather the signature from the multi-party on the reports. There is no problem to do this because the first level has laid a foundation for this.

The other is whether the system facilitates the usage of existing management information system that has been used by each participant to carry out the internal information management. The answer is also yes. To solve this problem, the relative participant needs to use simple script language, such as VB script, to develop small programs for extracting data from its own management information system and creating the required XML file by using the data. Then the user can avoid filling in the form when he submits the corresponding document. This is possible because the interface for submitting documents allows the usage of XML file and the schema of XML files are made open to the multi-party.

Now, the prototype system has been finished and it will be put into trial operation soon.

## 6 CONCLUSION

In this paper, the characteristics of the outward information management in construction projects in China are summarized based on literature survey and field investigation, and a Web-based approach to developing the system for outward information management of construction projects is proposed. Then the prototype system that we have developed based on the approach by using the latest information technologies including XML is examined and its applicability is discussed. The system can accommodate efficient collaborative work among the participants from multi-party in construction projects and facilitates each participant to grasp the related information effectively. In addition, the system facilitates the participants to make use of their own management information system with this system.

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# Visualizer:An Interactive Graphical Decision-Support Tool For Service Life Prediction For Asset Managers

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**Summary: In the past, practitioners performed service life prediction using subjective methods and heuristic knowledge. Although fraught with inaccuracies, these methods served industry well for a long time, to a point. Deterministic models were developed to assist practitioners calculate remaining service life; however, the results from these models are generalized and could not accurate predict the remaining service life in all individual cases. Markov chain modeling, employing actual service life data collected from field observations, has been used in specific domains, such as roofing systems, to simulate performance deterioration. Another example of Markov chain modeling, relying on expert knowledge, has been used to simulate deterioration profiles. Case-based reasoning is yet another way to calculate the remaining service life, basing the service life prediction on cases similar to the asset in question.**

**Establishing the service life of assets and their components is an essential ingredient of asset management. The service life must be established at time of installation of the asset and again at any time during the life of that asset. For many types of assets, the calculation or prediction of the remaining service life is difficult to accomplish, if not impossible. However, asset management cannot take place without knowing how long an asset or its components will continue to perform the intended functions. Furthermore, life cycle costs cannot be calculated without knowing the service life, and alternatives cannot be compared without knowing the life cycle costs. Moreover, maintenance optimization cannot take place without being able to compare the viable alternatives and their respective life cycle costs.**

**Visualizer is a prototype tool that has been developed to assist asset managers calculate the remaining service life of specific types of building assets and components. Visualizer is an interactive, graphical application that reads data from different sources and presents the accumulated data to decision-makers in a user-friendly format. Several of the methods for service life prediction have been implemented in Visualizer and are compared in the paper.**

**Keywords: Service Life, Asset Maintenance Management, Roofing Systems, Information Technology, Visualization**

## 1 INTRODUCTION

As organizations in the construction industry become more technologically efficient, as more companies become more information technology (IT) savvy and as data collection for condition assessment becomes more automated, there is an increasing need for IT frameworks for asset management. The proliferation of software applications in this industry, rapid obsolesence of computer hardware in business application and changing data protocols and operating systems in businesses today make it currently impossible to develop a centralized framework that can function over extended periods of time.

### 1.1 The design situation

Designers and specifiers in the construction industry are primarily concerned with a facility's required levels of functionality and may give some consideration to a specified design life. They must weigh the relative importance of various functional and performance needs that define the project but must also be aware that a system's capability to fulfil the requirements changes over the predetermined service life. The change of one component's performance relative to another is non-uniform; that is, the rate of deterioration of one component is often different from that of another. While a premature loss of function of one component does not necessarily mean the end of technical service life, it may drastically reduce the economic benefit associated with having and maintaining any one given system. Design therefore should be conducted with consideration of clearly identified service lives and with an awareness of the affect of the in-service environment.

### 1.2 The construction need

To effectively operate a portfolio of diverse facilities it is essential to have a firm understanding of the asset base, to know the operational and functional requirements as well as the associated financial implications, and to monitor the in-situ performance over lengthy periods of time (Vanier 2001).

As-built, and as-modified, drawings and owner/operators' manuals are basic to the understanding of 'in-service' facilities. The maintenance and inspection practices, and recording thereof, should be of a consistent format to readily permit periodic re-assessment of remaining service life. Time and material tracking should also be adequately detailed to permit the careful evaluation of the relative effectiveness of maintenance options.

Facility owners and operators maintain, repair, or replace their assets based on the budgets available, the operational priorities, the perceived economic advantage of maintenance options and their strategic plans for the property. The effectiveness of any (system level) rehabilitation scenario is related to:

- the technical performance and life expectancy of the proposed system;
- the expected and required service lives;
- the significance of the system to the value of the overall facility; as well as
- the present value (PV) of all associated costs (ASTM E917).

A maintenance, repair or replacement (MR&R) action should only be selected after careful examination and comparison of the life cycle economic implications of that action to feasible alternatives. And technical service life assessment is a pre-requisite to a life cycle economic evaluation.

### 1.3 The challenges and opportunities

There is considerable data required to make sophisticated decisions about even one asset; however, the problem is multiplied exponentially when hundreds of diverse assets are compared. How can asset managers of diverse asset unify their existing data, supplement them with corporate knowledge, and then use their data to make productive, cost efficient and effective decisions about MR&R?

## 2 THE INFORMATION DILEMMA

If one were to step back from immediate organizational challenges and take a detailed look at an overview of the existing and required information for making decision about MR&R, it reveals a whole spectrum of diverse formats and many different levels of detail in any of the existing data sets. This heterogeneity of data puts additional requirements on any information management systems.

### 2.1 Heterogeneity of data in the asset management domain

It is possible to distinguish three basic, although not always clearly distinct, categories of information:

(1) *Unstructured information* encompasses unstructured or barely structured text (e.g. ASCII, tab delimited and a variety of "non-text" information such as bitmap images, video, audio). Its content is meaningful to humans but not to machines, which can only read and handle such data at the low level of pixels or simple strings or as complete, fixed documents and display it in a single, fixed way. In fact, much of this type of data is still in hardcopy format and in a medium other than digital (e.g. photos, slides, or analogue tape).

(2) *Information with structured display* includes a variety of primarily textual information such as web pages, structured word-processor documents, or user queries, which are structured according to the HTML specification, a document model (data type definition or DTD), or a particular query syntax. The same category also includes some primarily visual information, for example CAD databases whose underlying schemas involve entities such as lines and shapes but not objects such as doors, columns and roofs. Different displays of such information are possible (e.g. using Cascading Style Sheets for HTML documents or changing layer settings in CAD drawings), but they involve only alternative display properties such as colours, fonts, or line types.

(3) *Information with structured content* implies machine-readable representation structured according to a certain conceptual schema of a domain (e.g. "smart" CAD drawings, some eXtensible Markup Language or XML documents, and databases). A multitude of essentially different human understandable representations can be derived from a single data set: a schematic 2D drawing, rendered 3D model, process simulation, or textual report, for example. These alternative displays can differ not only in display properties but also in modality (i.e. text vs. graphics vs. audio).

### 2.2 Current standardization efforts

As in many other domains, the need to minimize data re-entry and to enable horizontal and vertical interoperability has led to the adoption of product modeling and to industry-wide initiatives for the development of a standard domain model. Building upon the more general ISO STandard for the Exchange of Product Model Data (STEP 2001), the Industry Alliance for Interoperability (IAI 2000) is bringing together key industry participants from all around the globe. IAI is developing a standard schema for the architecture, engineering, construction and facilities management (AEC/FM) industries—the Industry Foundation Classes. An increasing number of software applications already feature IFC support: those developed by Nemetschek, Viatek Oy, Jidea Oy, and NAOKI among asset management software and numerous applications typically used in pre-operational phases. A list of IFC implementers is available at the IAI Nordic Chapter Web site (IAI Nordic 2000).

Another associated standardization effort— aecXML™ is focusing on data exchange over the Internet (aecXML 2000). The AEC/FM industries have acknowledged the importance of Internet-based communication for a domain relying on information from disparate sources and the potential of the eXtensible Markup Language (XML) standard (XML 2000). AecXML™ represents a framework for using XML for electronic communications in the domain.

### 2.3 Advantages and limitations of standard data models

The use of a standard data model enables communication between diverse applications as well as re-use of data entered by any participant in any phase of an asset's service life; thus ensuring data consistency and heightening reproducibility of analysis and assessment results.

However, it is becoming increasingly clear that the use of a standard data model is not an omnipotent solution for interoperability. Amor (2000) gives an overview of unresolved issues related to their implementation, some of which may be intractable. For example, along with dozens of practical and technical issues such as those related to business interests of information providers, data transport mechanisms, version management etc., there are potentially more serious conceptual questions related to sufficiency, validity and comprehensiveness of data models and mapping between different views. The work described in this Visualizer paper focuses only on those issues related to heterogeneous information representations.

Even if a model is accepted as an industry-wide standard, there will always be sub-domains, related domains, and applications that are better served by schemas with partly overlapping scopes and structures of their own. In addition, data required for the day-to-day operations of asset managers also include a variety of external data such as libraries and regulations as well as the company's business information (e.g. personnel files, accounting data). It is possible to distinguish three partially overlapping information spaces:

- global information space;
- project information space; and
- company information space.

An important difference between these three spaces is the possible level of control and management of the information contained. While the company information space can be fully controlled (i.e. theoretically all information can be structured in a standard way and be fully interchangeable), the global information space is driven by a variety of unpredictable free market forces and is hardly submissive to a single schema. This is also reflected in the project information space where it can be difficult to bring together all participants in a project using compatible systems and data representations.

Therefore any information system design in a domain such as asset management should take into account the unavoidable coexistence of multiple schemas, of multiple views within a single schema, of structured, differently structured, and unstructured information, and of document-based and model-based communication.

## 3 PROPOSED FRAMEWORK FOR DATA MANAGEMENT IN ASSET MANAGEMENT

The development of the framework for data management in the asset management domain starts from the assumption that Internet-based communication, distributed systems, and efforts on the development of IFCs and XML will continue into the foreseeable future. A proposed framework is presented here through a description and analysis of a prototype tool developed to demonstrate, test, and further explore the use and display of data for decision support tools for asset management. The goal of the work is to define a user-centered information architecture for asset management and hence the presentation layer represents the focus and the starting point.

### 3.1 Visualizer

Visualizer was developed within the Building Envelope Life Cycle Asset Management (BELCAM, 2001) project , a joint endeavour of the National Research Council Canada (NRCC), Public Works and Government Services Canada (PWGSC) and a number of North American partners, including the University of British Columbia (UBC). BELCAM was a three-year research and development project to investigate the integration of asset management technologies, namely: maintenance management, life cycle economics, service life prediction, and risk analysis. For the initial stage of BELCAM, work concentrated on "low slope" roofing systems as a "proof of concept" and therefore the prototype application operates only with data related to low slope roofs. NRCC, PWGSC, and UBC contributed to the development of Visualizer.

Visualizer is a prototype application that allows a user to ask relevent questions concerning their assets and to view the answers interactively. Visualizer is implemented in Visual Basic for Applications (VBA) in Microsoft Excel and uses Microsoft's Automation for integrating data entered and generated within different applications.

The business layer of Visualizer consists of formulae for the probabilistic prediction of asset performance, the assessment of risk and cost, and procedures for the multi-objective optimization of maintenance activities (Lounis *et al* 2000, Kyle *et al* 2002a). Data required for Visualizer to function include:

- visual representation (map) of the inventory,
- stored data about the inventory (mostly alphanumerical),
- external data (e.g. current maintenance costs), and
- user input.

### 3.2 Data entry and data display

The user interacts with the tool according to the following scenario:

1. *"FILES Menu":* The user imports a drawing as shown in Fig. 1, (FILES/INSERT GRAPHICAL IMAGE) of the building from any drawing software supporting Automation or creates a building outline in Visualizer using MS Office AutoShapes. The images can also be imported raster maps (BMP, GIF, JPG, etc.). Precise spatial information, that is—the exact location of buildings, is not vital for the application as these maps serve only for visualization purposes and relative positioning and therefore only simple rough sketches are needed.

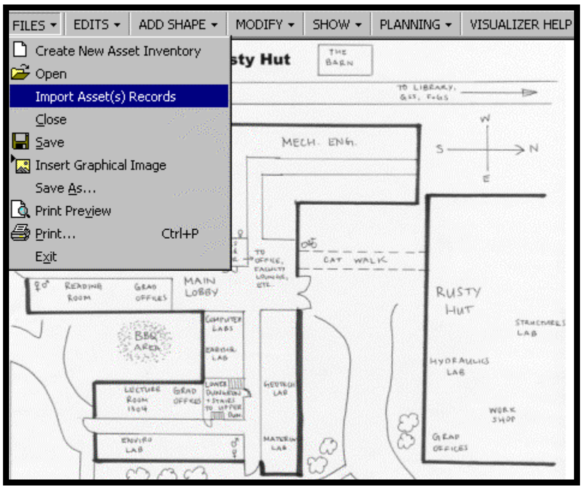



**Figure 1. Import option for images and files**

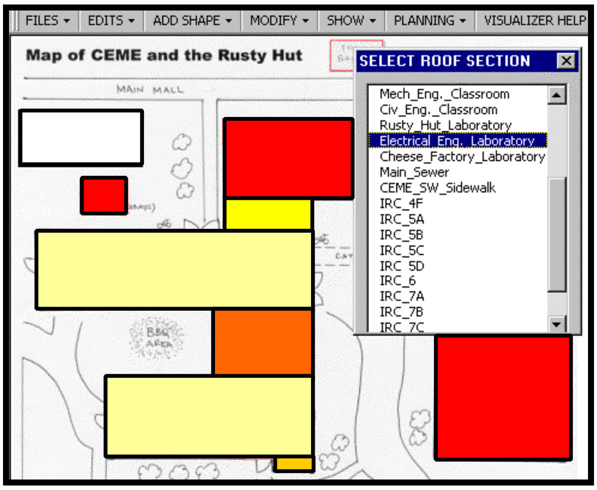


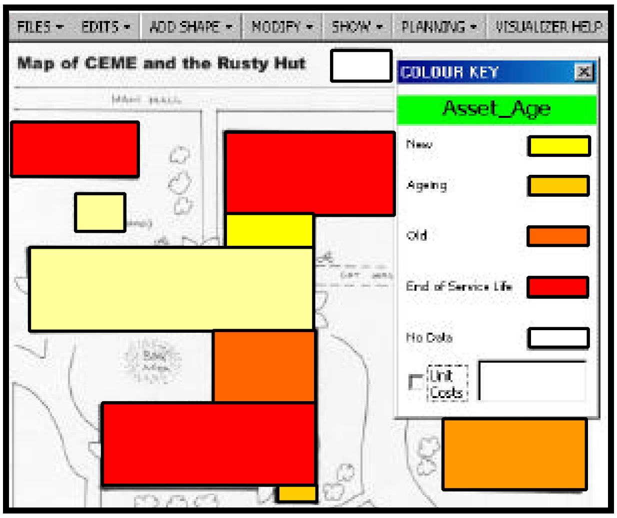

**Figure 2. Linking database records to graphics**

2. *"FILES Menu":* The user then imports the asset data (FILES/INSERT GRAPHICAL IMAGE) for the buildings, roofs, and roof sections from MicroROOFER (or potentially any other correspondingly structured database). MicroROOFER (2000) is an Engineered Management System for low-slope roofs from the US Army Corps of Engineers (Bailey *et al*, 1989), running under MS Access.

3. *"ADD SHAPE Menu":* The user can then draw polylines or rectangles representing discrete roof sections on the image (see the white rectangle in top left corner of Fig. 2). A wide selection of shapes are available to the user.

4. *"MODIFY Menu*": The user then links the polyline shapes to the appropriate roof sections from the corresponding database record using the dialogue box shown in Fig. 2.

5. *"SHOW Menu":* After these actions are complete, all data from the dataset can be visualized, i.e. properties of an object can be translated into the graphical language and represented as properties (colour, hatching pattern, etc.) of that object's surrogate (i.e. shape). Figure 3 displays the age of the roofing assets (shades of gray are displayed in this figure to represent the Visualizer colours of "red" for "End of Service Life" and "yellow" for "New"). Hatching patterns, as illustrated in Fig. 4, can be used to identify discrete data (i.e. types of roofing membranes, types of assets, building category codes).



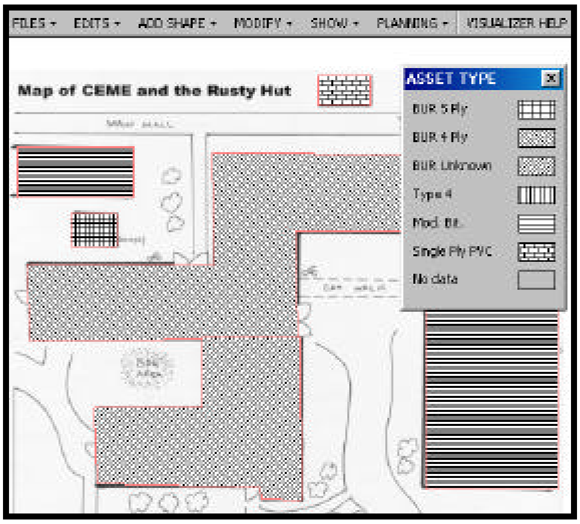
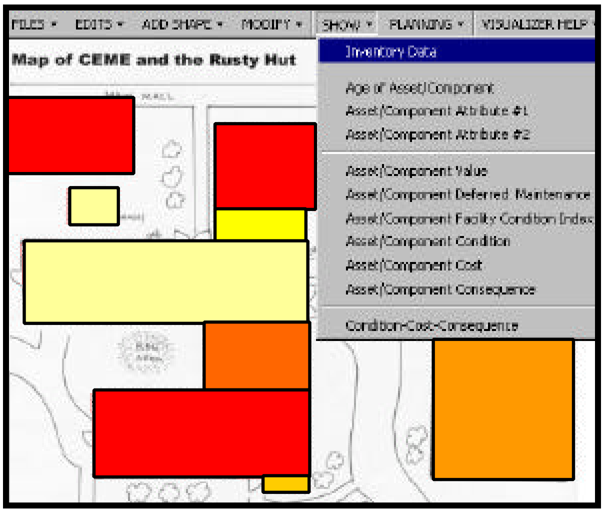




**Figure 3. Graphical display of continuous data.** **Figure 4. Graphical display of discrete data.**

6. *"SHOW Menu":* Figure 5 displays menu options about data imported from other datasets. For example, data about the "Six Whats" of asset management (Vanier 2001) can be viewed: "What is it Worth [Value]", " What is the Deferred Maintenance", or "What is the Condition". Figure 6 displays inventory data retrieved from the MicroROOFER dataset.



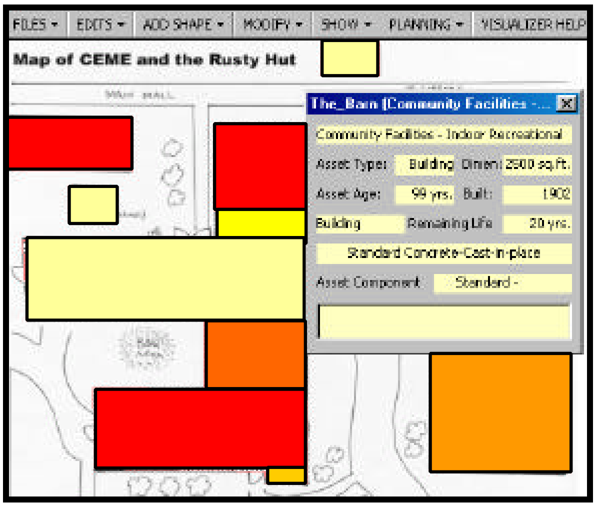




**Figure 5. Data display options** **Figure 6. Inventory MicroROOFER data.**

7. *"SHOW Menu":* In addition to displaying the existing roof condition as shown in Fig. 7, the application also can display the consequence of failure, as shown in Fig. 8. Please note the differences between Figs. 7 and 8 where one asset may be in poor condition but can have a low consequence of failure (roof at top left of images) and *visa versa* (building at bottom right).

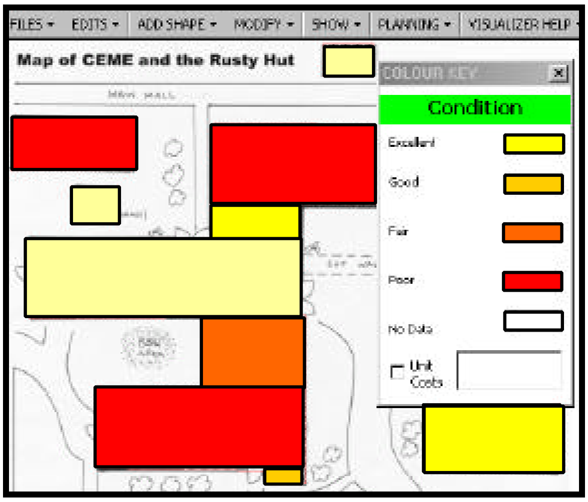



Figure 7. Display of condition data.

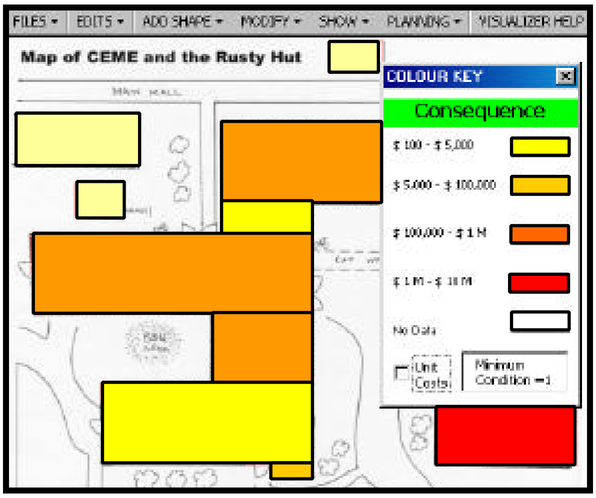



Figure 8. Display of consequence data.

8. *"SHOW Menu":* Individual managers can have different priorities when it comes selecting projects for MR&R. For example, some managers might be risk-averse; others can be cost-conscious, or some might be performance-oriented. As a result, these asset managers' decisions are based on their personal objectives and agendae. Figures 9 and 10 demonstrate the visualization of maintenance priorities using a simplified weighting of condition, cost and consequence. Figure 9 presents the maintenance priorities of a manager weighing condition, cost and consequence equally. Fig. 10 shows the different priorities of a manager equating cost and consequence, but not considering consequences of failure. As can be seen by comparing Figs. 9 and 10, the priorities will change when the weights on these three objectives are altered. Figures 9 and 10 do not show absolute values but provide the user with the relative priorities of MR&R for different roof sections.

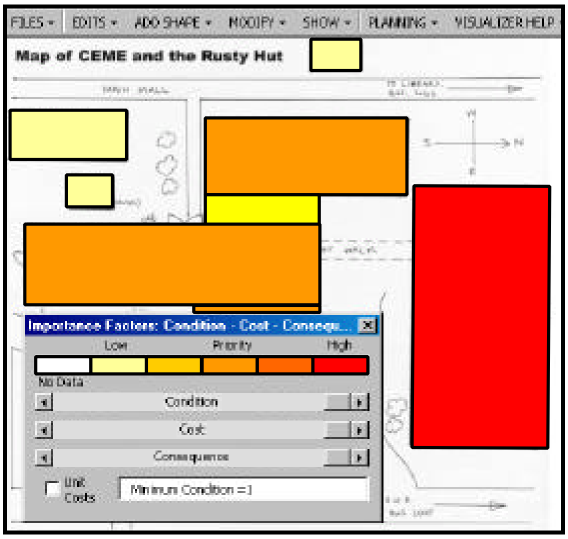



Figure 9. Condition = cost = consequence.

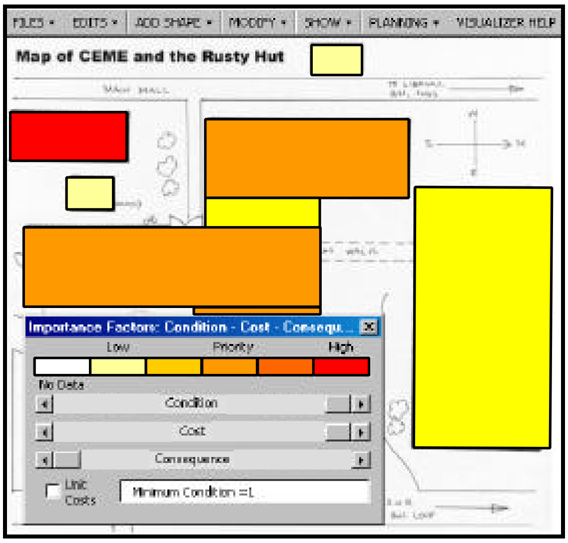



Figure 10. Condition = cost, consequence = 0.

### 3.3 Service life models

The user can link service life models to the polyline shapes as shown in Fig. 11 (PLANNING/LINK SERVICE LIFE MODEL). Currently the four different service life models for low slope roofing systems are Simple, Expert, Deterministic and Field Data, as shown in Fig. 12.

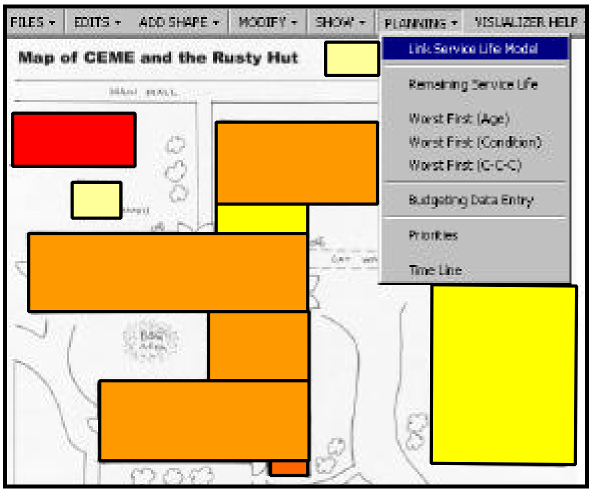



Figure 11. Planning models.

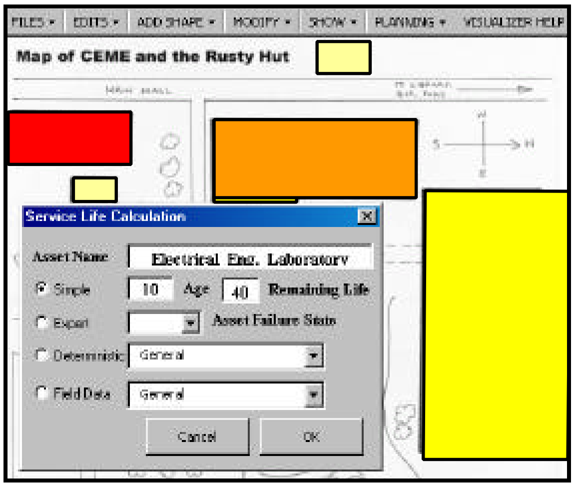



Figure 12. Linking service life model

*Simple:* For this service life model, the age of the asset is calculated from data provided (date of construction or date roof was replaced); the user can override these data and can input a number for the remaining service life, as shown in Fig. 12. The condition of the roof in any future year will be directly proportional to the remaining service life at that time (straight-line deterioration).

*Expert:* In the Expert Planning Model, the user first selects the "Asset Failure State" as shown in the dialogue box in Fig. 12. The condition of the roof in any future year will be proportional to the remaining service life at that time (exponential deterioration).

Determistic: In the Determistic Planning Model, the user selects the service life model used in the original MicroROOFER software.

*Field Data:* In the Field Data Planning Model, the user selects the service life model developed using the Markov Chain described in the associate paper (Kyle *et al* 2002).

### 3.4 Remaining service life

After the service life models have been linked to the polyline shapes, the remaining service life can be calculated and displayed (PLANNING/REMAINING SERVICE LIFE). Figure 13 displays the results of the calculations from the service life models.

### 3.5 Planning Models

Figure 14 displays one of the planning models currently implemented: the "Worst First" option has been selected (PLANNING/WORST FIRST). In this option, all the roof assets that have a Condition State of less than 3, as selected in the Fig. 13 dialogue box, are raised to Condition State 3 (Kyle *et al* 2002) and the cost of MR&R are calculated and displayed.

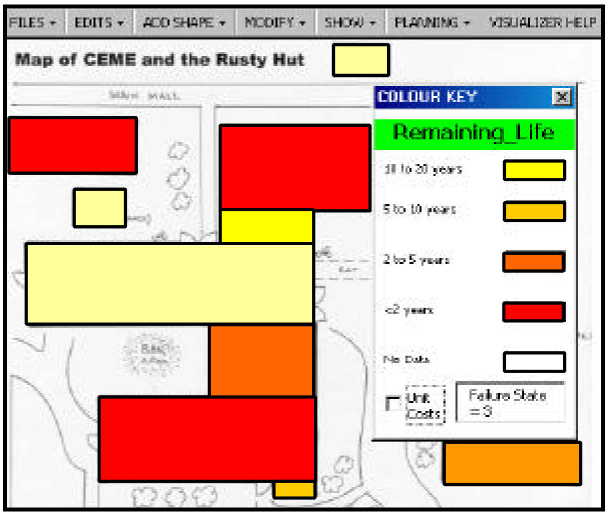



Figure 13. Display remaining service life.

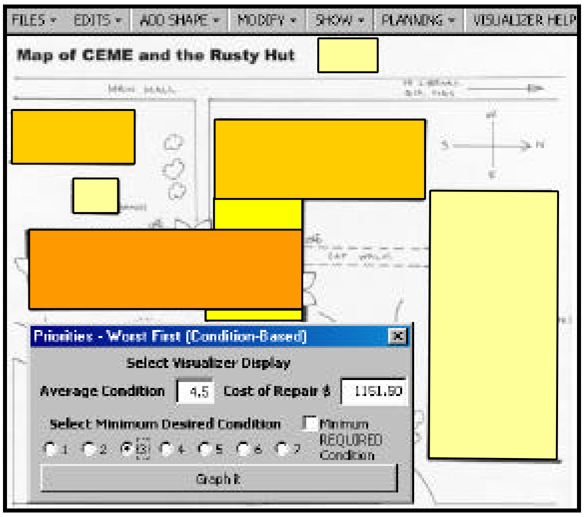



Figure 14. "Worst first" planning model.

## 4 DISCUSSION

In this section observations made during the development and testing of Visualizer are discussed, particularly in the light of the previously stated assumptions about the future of data communication in the domain of asset management. These observations were not scientifically tested, measured, and verified at this stage but should serve as starting guidelines for the future research and development.

### 4.1 General observations

- Visualization of natively non-visual data for large asset inventories can be a highly useful cognitive aid for grasping the overwhelming amount of information required for decision support in asset management.
- Visualization tools can easily be developed where conceptually structured information is available.
- Data internally represented in terms of objects or entities defined by sets of attributes or properties and their values can always be displayed in a variety of ways with entity surrogates (representations) and their attributes serving as carriers of information about the entities.
- Basic visualization and analysis techniques have already been developed within GIS and these types of visualization should be extracted, adapted and incorporated into asset management tools.

### 4.2 Implementation

Visualization or display translation can be implemented as a service that can form part of many diverse applications, on both clients and data servers (Froese *et al*. 2000). Along with information about objects, their properties and property values, the data layer needs to expose information about data type and data range for each of the properties, while the presentation layer needs to make available information about its expressive vocabulary.

### 4.3 Opportunities

An important feature of information management tools is the ability to work at a varying level of detail; this is one of the most important techniques in GIS, usually called—generalization. As a domain schema identifies generic and hierarchical relationships between concepts it can be used as a possible tool for facilitating navigation between different levels of detail.

Another GIS technique—information overlay, which is crucial for exploration of interdependencies between data, can also benefit from model-based exchange. The ability to combine and overlay properties from disparate sources depends on the availability of such data in the common format.

Visualizer was easy to implement thanks to the power of Microsoft's Automation. However, most pieces of data required several translations—in addition to translation between different modes (alphanumerical to graphic), one human-readable text string often needed a translation for each application (e.g. elimination of white spaces or particular special characters). This requirement coupled with the need to enable navigation between different levels of detail resulted in a number of mapping tables and lines of code that might be avoided with the use of standard data models.

The weakest point in the application is the need to associate shapes with database records—it requires user's effort and the need for solutions related to data updating and consistency. This need can be eliminated in model-based systems or by using integrated CAD databases.

Model-based systems also allow generically different representations of entities themselves (e.g. 3D models or a variety of symbolic representations) thereby providing a richer vocabulary for translations.

### 4.4 Implications for further research

One of the goals of the research is to develop a graphical user interface that would allow direct interaction with all asset-related information. At the same time, the interface would have to:

- Maximize access to data,
- Optimize display, and
- Minimize storage space, more precisely, repetition of same information and the accompanying risk for errors and inconsistencies.
- Further steps towards attaining this goal are as follows:
- Re-examination of the data that should be archived, referenced, or discarded at the commissioning of a facility in an Internet-based, distributed environment;
- Supporting and/or contribution to the further development, implementation, and acceptance of the IFCs, aecXML, XML, and related standards;

- Design of generic translation services;
- Investigating the potential of the aids and services for both enhancing information retrieval and facilitating translation into alternative displays; and
- Implementing the aids and services transparently in user interfaces allowing alternative access structures to project information including but not limited to a 3D geometric representation of the facility.

## 5 CONCLUSIONS

Standardized domain models are undoubtedly a rudimentary answer to the efficient data management in an information-wise and complex domain such as asset management. However, these models should be developed, advocated, and used with the awareness that no schema can cover an entire domain in full detail, or from each and every viewpoint, and that there will always be data and processes that are not submissible to it. On the other hand, standard models should be considered not only as tools to enhance existing systems but also as an opportunity to redefine data use in the domain.

The paper has examined the visualization techniques available and demonstrates how asset managers may use Visualizer to compare, and "see", the impacts that different service life prediction models have upon their asset management plans. Addition research is continuing in this area (NSERC 2001).

The paper stresses the importance of formalized information storage, updating and integration in the domain of “Service Life Asset Management”. The key aspects of information storage and flow in this domain, identified in this paper, are: (1) the data needs for decision support tools, (2) the development of quality metrics for evaluation of performance and condition; (3) the need for standardization of data and information flow, and (4) the need for continuous data transfer amongst actors in the process. The authors also identify the need for continued research and standardization in all these domains. This paper is exploratory in nature and encourages the reader to visualize what data are required for “Service Life Asset Management” and how those data can be used.

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# Attitude And Behaviour In The Maintenance Management Environment: A Solution Focused Approach

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**Summary: The general aim of this study was to explore the relationship between perceived control in the work environment and effectiveness of information technology (IT) in a computerized maintenance management environment. The study follows on from previous work on the perceptions of efficiency of training and information technology, the effect of information technology on human relations within the work place and the perceived impact of information technology on the efficiency of occupational performances as a function of position, age, experience and gender, over time regarding the use of a computerized maintenance management system in a large public sector asset management organization in Australia. The analysis of the data indicated that staff perceived that the computerised management maintenance system reduced their control over the work environment (deindividuation) and it is considered that this leads to a lowering of motivation and learned helplessness in the work environment. Cognitive intervention could be used in an attempt to integrate IT usage with perceived ownership and control over the work environment. It is further considered that this could be achieved by coaching, that is coaching the human resources managers to enable them to train their staff in the use of computerised management maintenance systems, leading to a possibility of a reduced sense of alienation in the work place that may have the possibility of leading to improvements in productivity in accordance with the surveys and the analysis of the data that has taken place in this work environment to date. Further research work could be undertaken on coaching or on other research topics in fundamentally important areas to building, with regard to IT.**

**Keywords: I.T., perception, productivity, asset-management, coaching**

## 1 INTRODUCTION

"Life is change-and always has been-but now the world is changing at an unprecedented rate. … with change come both opportunity and threat." (Grant & Greene 2001 pp. 2). The continual changes in the work environment produce changes to the perceptions and consequently attitude and behaviour of personnel. I.T., being the fifth paradigm shift in the industrial revolution identified in previous research (Mathews 1994), appears to be a principal cause of change within the workplace. I.T. is considered by researchers and practitioners to be extremely useful in terms of increasing both qualitative and quantitative outcomes (Mathews 1994). "Information technology is having pervasive effects throughout the economy that are entirely analogous to the previous paradigm shift" (Mathews 1994 pp.84). This paradigm shift is continually transforming a plethora of existing organizations within the building construction industry and generally. ESRC, CRIC and PREST, in their final report in July, have reported that the Esprit program recognised the area of human-computer interaction as an important generic activity (ESRC Centre for Research in Innovation and Competition (CRIC) and Policy Research in Engineering, Science and Technology (PREST), 2001 pp. 61)

"Like it or not, change cannot be turned off. At the moment it is flowing uncontrollably. Put your hand over it and the water will spread in all directions. Sit back and you will drown" (Ridderstrade & Nordstrom 1999). Whilst change may be considered to be desirable for a plethora of different reasons it may also have the potential to cause some undesirable effects. "Change can be wonderful and exciting, but it can also instill fear and anxiety" (Zeus % Skiffington 2000 pp.198). Zeus and Skiffington purport that we fear "what the future may bring in the workplace" and that "new technologies and new procedures may make us redundant or at the least appear incompetent" (Zeus & Skiffington 2000 pp.198). The worst-case scenario may be that "We fear that change will destabilize us because we do not have the energy or strategies to cope with new demands" (Zeus & Skiffington 2000 pp.198). Change is difficult because it means an alteration in our world and "… it is the most common situation causing anxiety and feelings of helplessness because it always means some disruption of ties or relationships" (Zeus & Skiffington 2000 pp.199). People naturally resist change and individuals vary in their response to change. Literature suggests that the process we go through when adapting to change is similar to what we experience when dealing with loss and involves four stages: denial, resistance or anger, emerging acceptance and finally commitment to change. In research on change by Chisthi, Martin and Jacoby it was claimed that a "negative effect on the mental health and morale of employees was reported by a noticeable percentage of respondents" (Chisthi et al 1997 pp. 11). This claim supports the 'resistance to change' theories because organisational change may force managers and support staff alike to move from their comfort zone and may impact on their mental health (Lansbury 1986). "The technology changes so fast that unless they continually retrain they cannot hope to keep up" (Grant & Greene 2001 pp. 3). This creates a paradoxical situation where on the one hand we have to be able to change and on the other hand we want and need the certainties that our parents and grandparents had. "Many of us are afraid of change, afraid of the future. We would rather stay with what we know, with what is safe, even if it is making us unhappy." (Grant & Greene 2001 pp. 4). From an organizational psychology perspective, it is interesting to consider the implications of these continual changes on factors within a maintenance management environment that has undergone a substantial change, the introduction of a new I.T. system, within the past year (from the time of the data collection). Change necessitates choice and that produces conflict that culminates in anxiety in the people involved. Mathews purports that most programs of organizational change end up in failures. Children are most adept at change, as we grow older it becomes more difficult to change. Often we want to change habits and sometimes seek help but so often, despite all this effort the behaviour stays with us (Grant & Greene 2001 pp. 4). Research has identified the following factors of change agents: communication effort, orientation, empathy and credibility and since several of these are pertinent to the role of coaching it could be argued that "A coach is a catalyst for change" (Zeus & Skiffington 2000 pp.201). Coaching is simply using the will to change, taking the first step and setting off in the direction we want to go, then planning very carefully the pathway that we will lead to where we would like to be. "This means working out what our values are, what is important to us and what we want to spend our time and energy doing" (Grant & Greene 2001 pp. 14).

In the previous studies, "Analysis of Phenomenological perceptions of effectiveness of information technology in computerised maintenance management" (Clarke and Clarke 1999), "Effectiveness of I.T. in computerised maintenance management: a longitudinal study of the analysis of phenomenological perceptions"(Clarke 2000), "Effectiveness of IT in computerised maintenance management: a longitudinal study of the analysis of phenomenological perceptions" (Clarke 2001), "Perceptions of the effect of I.T. on training, human relations and productivity as a function of position, age, experience and gender" (Clarke and Clarke 2001) and "A longitudinal study into perceptions of the effect of I.T. on training, human relations and productivity in the construction industry as a function of position, age, experience and gender" (Clarke and Heathcote 2001) data was collected and analysed to assess the impact of technological change in terms of training, human relations and productivity as a function of position, age, experience and gender. This study was intended to explore the relationship between perceived control in the work environment and effectiveness of IT in a computerised maintenance management environment or more specifically to explore the possibility of linking cognitive intervention to improved perception of IT in the workplace and hence improvement in productivity

## 2 METHOD

The data utilized in this study was collected during two previous studies. The survey framework included an interview survey technique within the organization in the technological field to gain more qualitative information on strengths and trends in the sector (ESRC Centre for Research in Innovation and Competition (CRIC) and Policy Research in Engineering, Science and Technology (PREST), 2001, pp. 64). The initial set of data collected was for the study entitled "Analysis of phenomenological perceptions of the effectiveness of information technology in computerized maintenance management" (Clarke and Clarke 1999). This data collected for the 1999 study, (N=34), was categorised into the broad headings of training, human relations and productivity and was differentiated by position and in the 2001 study by Clarke was further analysed by differentiating by age, experience and gender. Where the positions were either asset managers or support/ancillary staff and subjects with age of 40 years or greater were categorised as being older subjects, whilst subjects with at least ten years of experience of more were considered experienced. The study (Clarke and Clarke 1999) developed a framework technique together with procedures for statistical analysis that have been replicated in the following studies "Effectiveness of I.T. in computerized maintenance management: a longitudinal study of the analysis of phenomenological perceptions" (Clarke 2000), "A longitudinal study into perceptions of the effect of I.T. on training, human relations and productivity in the construction industry as a function of position, age, experience and gender" (Clarke and Heathcote 2001) and "The effect of IT on staff perceptions: observed differences and outcomes of studies conducted within the maintenance management environment in Australia" (Clarke and Clarke 2001). The second set of data was collected for the study entitled "Effectiveness of I.T. in computerized maintenance management: a longitudinal study of the analysis of phenomenological perceptions" (Clarke 2000). This set of data, (N=53) was collected a year after the introduction of a new I.T System was introduced into the organization being observed and was analysed by differentiating by position. The second data set was further analysed by means of differentiation by age experience and gender in the study "A longitudinal study into perceptions of the effect of I.T. on training, human relations and productivity in the construction industry as a function of position, age, experience and gender" (Clarke and Heathcote 2001). The two data sets were tabulated to facilitate analysis by t-tests to permit observations of differences, similarities and changes, refer to tables 1p, 1a, 1e, 1g, 2p, 2a, 2e and 2g.

## 3 RESULTS

The results used in this study were derived in the studies “Analysis of Phenomenological perceptions of effectiveness of information technology in computerised maintenance management” (Clarke and Clarke 1999), “Effectiveness of I.T. in computerised maintenance management: a longitudinal study of the analysis of phenomenological perceptions”(Clarke 2000), “Effectiveness of IT in computerised maintenance management: a longitudinal study of the analysis of phenomenological perceptions” (Clarke 2001 ), “Perceptions of the effect of I.T. on training, human relations and productivity as a function of position, age, experience and gender” (Clarke and Clarke 2001) and “A longitudinal study into perceptions of the effect of I.T. on training, human relations and productivity in the construction industry as a function of position, age, experience and gender” (Clarke and Heathcote 2001). Following the data collection phase of the studies the data was analysed. The unpaired t-test procedure was used with independent statistical analyses being conducted to control the type one error at 0.05.

Results of the analysis of the initial data set, (N=34), concerning training, human relations and productivity as a function of IT and as differentiated by position were obtained from the study by Clarke and Clarke, 1999 and selected human relations results are as shown below. Results for office morale as a function of IT as differentiated by position were $t = 2.25$, $p< 0.05$ and a mean score of 4.167 for managers and 3.588 for support staff, a statistically significant difference. Perceived control in occupational performance as a function of IT as differentiated by position had test results of $t= 2.604$, $p< 0.05$ with a mean score of 4.167 for managers and 3.235 for support staff, a statistically significant difference. Fear of redundancy as a result of IT as differentiated by position had a test result of $t = 4.243$, $p < 0.05$ and a mean score of 3.882 for managers and 3.0 for support staff, a statistically significant difference, refer to table 1p.

Selected results of the analysis of the initial data set concerning training, human relations and productivity as a function of IT and as differentiated by age are shown below. The differentiation with age assumes that a person less than 40 years is categorized as being young and a person equal to or greater than 40 years of age is categorized a being old, for the purposes of this study. Perceptions of I.T.’s effect on office morale as differentiated by age revealed test results of $t = 0.649$, $p> 0.05$ and a mean score of 3.429 for younger subjects and 3.625 for older subjects. Perceived control in occupational performance as a function of IT and as differentiated by age revealed test results of $t = 2.5$, $p< 0.05$ with a mean score of 3.714 for younger subjects and 3.375 for older subjects, a statistically significant difference. The test results for the fear of redundancy as a result of IT as differentiated by age were $t = 2.097$, $p< 0.05$ and with a mean score of 4.143 for younger subjects and 3.438 for older subjects, a statistically significant difference, refer to table 1a.

Selected results of the analysis of the initial data set concerning training, human relations and productivity as a function of IT and as differentiated by experience are shown below. Experience is based on a minimum of ten years, so the subjects with ten years experience or more are considered to be experienced while those with less than ten years experience are considered inexperienced. Perceptions of I.T.’s effect on office morale as differentiated by experience had test results of $t = 0.98$, $p> 0.05$ and with a mean score of 3.333 for inexperienced subjects and 3.636 for experienced subjects. Perceived control in occupational performance as a function of IT as differentiated by experience had test results of $t = 3.394$, $p < 0.05$ with a mean score of 3.333 for inexperienced subjects and 4.272 for experienced subjects, a statistically significant difference. Fear of redundancy as a result of IT, differentiated by experience, the test results were $t = 0.539$, $p> 0.05$ with a mean score of 3.667 for inexperienced subjects and 3.833 for experienced subjects, refer to table 1e.

Selected results of the analysis of the initial data set concerning training, human relations and productivity as a function of IT and as differentiated by gender are shown below. Perception of I.T.’s effect on office morale, differentiated by gender, the test results were $t = 4.588$, $p < 0.05$ and with a mean score of 2.667 for female subjects and 3.75 for male subjects, a statistically significant difference. Perceived control in occupational performance as a function of IT, as differentiated by gender, the test results were $t = 6.114$, $p< 0.05$ and with a mean score of 3 for female subjects and 4.375 for male subjects, a statistically significant difference. Fear of redundancy as a result of IT, as differentiated by gender, the test results were $t = 1.015$, $p > 0.05$ and with a mean score of 4 for female subjects and 3.667 for male subjects, refer to table 1g.

These results were tabulated as were the results from the later data set, (N=53) and these are shown below in tables 1p, 1a, 1e, 1g 2p, 2a, 2e and 2g..

**Table 1p. Initial data set results as differentiated by position for human relations**

| **description** | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **POSITION** | | | | manager | support |
| Human resources | Office morale | 2.25 | <0.05 | 4.167 | 3.588 |
| Human resources | Control | 2.604 | <0.05 | 4.167 | 3.235 |
| Human resources | Fear of redundancy | 4.243 | <0.05 | 3.882 | 3 |

**Table 1a. Initial data set results as differentiated by age for human relations**

| **description** | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **AGE** | | | | younger | older |
| Human resources | Office morale | 0.649 | >0.05 | 3.429 | 3.625 |
| Human resources | Control | 2.5 | <0.05 | 3.714 | 3.375 |
| Human resources | Fear of redundancy | 2.097 | <0.05 | 4.143 | 3.438 |

**Table 1e. Initial data set results as differentiated by experience for human relations**

| **description** | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **EXPERIENCE** | | | | inexp | experienced |
| Human resources | Office morale | 0.98 | >0.05 | 3.333 | 3.636 |
| Human resources | Control | 3.394 | <0.05 | 3.333 | 4.272 |
| Human resources | Fear of redundancy | 0.539 | >0.05 | 3.667 | 3.833 |

**Table 1g. Initial data set results as differentiated by gender for human relations**

| **description** | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **GENDER** | | | | female | male |
| Human resources | Office morale | 4.588 | <0.05 | 2.667 | 3.75 |
| Human resources | Control | 6.114 | <0.05 | 3 | 4.375 |
| Human resources | Fear of redundancy | 1.015 | >0.05 | 4 | 3.667 |

**Table 2 p. Secondary data set results as differentiated by position for human relations**

| **description** | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **POSITION** | | | | manager | support |
| Human resources | Office morale | 0.201 | 0.8411 | 3 | 2.95 |
| Human resources | Control | 2.151 | 0.05 | 4 | 3.44 |
| Human resources | Fear of redundancy | 0.943 | 0.3499 | 3.1 | 3.9 |

**Table 2 a. Secondary data set results as differentiated by age for human relations**

| description | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **AGE** | | | | younger | older |
| Human resources | Office morale | 0.227 | 0.8215 | 2.913 | 2.969 |
| Human resources | Control | 0.986 | 0.3287 | 3.478 | 3.719 |
| Human resources | Fear of redundancy | 1.183 | 0.2421 | 3.522 | 3.188 |

**Table 2 e. Secondary data set results as differentiated by experience for human relations**

| description | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **EXPERIENCE** | | | | inexp | experienced |
| Human resources | Office morale | 0.099 | 0.9219 | 2.972 | 2.947 |
| Human resources | Control | 0.485 | 0.6295 | 3.611 | 3.737 |
| Human resources | Fear of redundancy | 0.519 | 0.6058 | 3.361 | 3.211 |

**Table 2 g. Secondary data set results as differentiated by gender for human relations**

| description | | t | p | Mean | scores |
|---|---|---|---|---|---|
| **GENDER** | | | | female | male |
| Human resources | Office morale | 2.165 | 0.0349 | 2.619 | 3.118 |
| Human resources | Control | 2.014 | 0.0491 | 3.286 | 3.794 |
| Human resources | Fear of redundancy | 0.677 | 0.5013 | 3.19 | 3.382 |

## 4 DISCUSSION

The first data set was analysed using a t-test and resulted in 16 of the 28 tests, 57%, displaying a statistically significant difference whereas the second data set resulted in only 4 out of 28 tests, 14%, being significant, with the type one error being controlled at 0.05. This may have been attributable to the introduction of a new computerized maintenance management system that tended to unify the work force, as a whole, comprised of the groups examined in the studies: manager, support, younger, older, inexperienced, experienced, female and male staff. On comparison of the two data sets it was observed that with the initial data set the results for the control of occupational performance were significantly different when comparing position, age, experience and gender, representing 100% of the variables, whilst with the second data set the significant differences were only observed for position and gender, representing only 50% of the variables. It is considered that this reinforces the earlier comment that the workforce is becoming more unified or the differences between the groups are considerably lessened perhaps by the introduction of the new maintenance management system. The analysis of the data revealed that the workforce, as a whole, perceived that they had a lower level of office morale and a reduced level of control over their work environment.

A summary of these results revealed that there was a higher perceived level of satisfaction with IT by managers as opposed to support staff but that this level of satisfaction was reduced substantially after the new I.T. system was introduced. Similarly younger staff was slightly more satisfied with I.T and there was a 13% reduction in this level of acceptance with the new system. Average mean scores, as differentiated by experience, revealed that between the first and second data collection there was an 86% reduction in the perceived level of satisfaction with I.T. by experienced staff. This was considered to be indicative of the loss of control within the work environment being experienced generally by the workforce. Finally male staff members were more satisfied with I.T. than female staff members, as indicated by the average mean scores by males being higher in 84% of the variables, over both data sets. Trends in the perceptions of the effectiveness of I.T. in the maintenance management environment in Australia have been observed to change in the time between the collection of these two data sets, indicating a reduction in the perceived level of satisfaction with office morale and control in the work environment.

General staff perceptions of I.T were reasonably high in terms of productivity and the empirical results were in line with the theoretical hypotheses derived from previous research including most of the results observed in the other studies by Clarke. Perceived control in occupational performance was observed to have significant difference between both managers and support staff and female and male subjects whereas differences in levels of satisfaction with the effects on office morale were only statistically significant between the groups differentiated by gender. The level of office morale is directly related to service and so customer satisfaction. There were some observed differences in the perceptions of staff, differentiated by position and gender, regarding control in occupational performance and office morale. This in itself may warrant further research to identify possible links between control and both position and gender that would influence customer relations.

In conclusion, it is evident from the present research that the maintenance management environment and the building construction industry in general, while perhaps more efficient quantitatively, has yet to come to terms with the impact on human relations of implementing new I.T. systems. It would seem that organizational factors are not being adequately addressed within this environment, perhaps as a consequence of the focus on technology as “the answer” at the expense of the staff involved in enabling this technology to operate. In particular, the results indicate firstly dissatisfaction with control of the work environment and secondly office morale. It is considered that deindividuation would lead to a lowering of motivation and then learned helplessness, in the workplace. It is further considered that cognitive intervention could be implemented to create an environment in which it would be possible to reverse this apparent trend and foster the perceived ownership of the IT system and hence perceived control within the work environment. It is further considered that this outcome could be achieved by coaching. As an organizational psychology issue, the researchers argue that it may be extremely beneficial to focus on an issue far broader than the specific environment focused upon in these studies. Within the construction industry in general, an examination of the impact of coaching on staff perceptions of I.T. Additionally a study of Customer Relationship Management (CRM) policy and effectiveness could be carried out as a function of the interaction between staff and the I.T. systems implemented. The researchers argue that given the findings of the present study, more attention needs to be given to the impact of I.T. on staff. Examining the construction environment and I.T. in the light of organizational psychological principles can do this. Implications of this proposed further research are argued to benefit staff satisfaction levels and ultimately productivity levels and CRM in general.

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# An Architecture For Extension And Reuse Of Standard Data Models In The Scope Of The Building Product And Process Life-Cycle

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**Summary: During the last decade many projects have been releasing significant results in the development of standard models and open interoperable platforms to support some of the major building and construction activities. Nowadays, several standards are already available and others are in advanced stages of development. However, it is expected that proposals for the development of new models will come due to the continuous need for the applications to extend their industrial scope. The increase in the number of models, most often developed using different methodologies and languages, has seriously contributed to the major difficulties found when examining interoperability between data models. Specifically for this industrial sector, the integration of libraries of components and materials is a necessity in the advent of the Internet and electronic data exchange. Therefore, it is of major importance to push suppliers to describe their product catalogues in a standard format. Should this be achieved, intelligent tools that adopt these standards could have a major contribution towards the construction of the "better buildings of tomorrow". This paper describes a proposal of a general architecture for standard-based integration platforms, supporting the reuse and extension of already existing standard data models and interfaces, including integration with standard-based libraries of materials and components.**

**Keywords: Interoperability, Reusability, Meta-Models, Mapping, Library of Materials and Components**

## 1 INTRODUCTION

The development of standard-based product and process data models to support the life cycle activities for the Building and Construction (B&C) industry has been addressed intensively during the last decade. In Europe for example, several projects worked on this issue, proposing and developing frameworks to assist in the establishment of interoperable open environments for the B&C industry.

The major projects of the early 90's that were the first to research and develop standard-based methodologies and integrated platforms for data exchange, share and archive in B&C domains, can be listed as follows: 1) the COMBINE project (European JOULE programme), developed a computer-based integrated building design system and demonstrated the feasibility of technology proposed by STEP (ISO10303 – STandard for the Exchange of Product model data) to integrate applications within the building field through a common integrated data model (COMBINE,1995); 2) the ATLAS project (European ESPRIT III programme), defined a methodological framework and software tools conforming to STEP to enable the integration of large scale engineering applications (ATLAS,1995); and 3) the RoadRobot project (European ESPRIT III programme), developed a STEP-based reference model and a multi-level integrated platform to automate the road construction activities from design to paving (RoadRobot,1995)(Pimentão,J.P.,1995). Nevertheless, the BRITE-EURAM CIMTOFI project designed and developed, between 1990-1993, one of the first prototypes and toolkits adopting what was in that time the emergent STEP technology (CIMTOFI,1993).

During the second half of the 90s, the VEGA project (ESPRIT IV programme) presented a proposal to bridge the gap between four standards (STEP, SGML, CORBA, EDI) developing the COrba Access to STep models (COAST) platform and services for distributing CAD applications (VEGA, 1999).

During the same period, the ESPRIT 25559 Project: "Supply Chain Management in Construction Industry – SUMMIT" developed an Electronic Data Interchange (EDI)-based communication infrastructure between various partners involved in the manufacturing and construction of prefabricated houses, intending to automate the tendering, ordering, delivery, invoicing and payment processes, in a context of heterogeneous software systems. The adopted methodology focused on the seamless

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integration of the most advanced technologies as IAI/IFC, STEP and EDIFACT standards (SUMMIT,1998)(Jardim-Gonçalves,R.2000b).

Recently started, the Electronic Business in the Building and Construction - eConstruct (IST-1999-10303), project aims to develop, implement, demonstrate and disseminate a new Communication Technology for the European Building and Construction industry, called Building and Construction (BC) eXtensible Mark-up Language (bcXML). This Communication Technology intends to provide the Building-Construction industry with a powerful but low cost communication infrastructure on the Internet (e-Construct, 2000)(Leonard, D. 2000).

Also, the project “C-ECOM: Cluster for Electronic Commerce” is an active concerted network sustaining a group of projects working in standardisation activities in the frame of research and technological development in the area of “New methods of work and electronic commerce”. C-ECOM is showing that standards are necessary to enable organisations to trade e-globally (CECOM,2001). Also, (Filos,E.,2000) in “Moving Construction towards the digital economy” shares the same opinion.

Starting at the end of 2001, the ProDAEC project is an IST supported “European Network for the Promotion of the Standards and eBusiness in the AEC sector”. ProDAEC’s main objective will be to set up and sustain a Network for the AEC sector that promotes the use and implementation of standards and best practices regarding product data exchange, e-work and e-business. The project will bring together material providers, construction companies, suppliers, designers, industrial federations, engineering consultancies, software vendors, R&D centres and universities.

## 2 STANDARDISATION EFFORTS FOR BUILDING AND CONSTRUCTION

The major results from the projects presented in the previous section pointed out that the adoption of normalised methodologies and platforms to achieve an adequate level of integration of applications and interoperable open environments would be indispensable. Primarily, the development and adoption of standard models for representation of the data structures was identified as the key to sustain the core of the integrated systems.

In an effort to provide an answer to these requirements, the TC184/SC4 (Industrial automation systems and integration - Product data representation and exchange: Industrial Data) of the International Organization for Standardisation (ISO) launched, within its WG3 (Product Modelling), the T22: Building Construction Group. Under the umbrella of T22, it was developed for ISO10303-STEP (STEP-1)(Introducing STEP,1999), the part 225 titled: "Application Protocol (AP): Building Elements Using Explicit Shape Representation" (SOAP,2000). This part is now an International Standard (IS) and specifies the requirements for the exchange of building element shape, property, and spatial configuration information between application systems with explicit shape representations, specifically the physical things of which a building is composed, such as structural elements, enclosing and separating elements, service elements, fixtures and equipment, and spaces (Haas,2000). Nevertheless, other parts of STEP have been developed contributing to the release of standard models related to the building and construction industry, e.g., AP228 (ISO 10303-228) Building services: Heating, Ventilation and Air Conditioning (HVAC) and AP230 (ISO 10303-230) Building structural frames: Steelworks (SOAP, 2000).

Also in Europe, the European Committee for Standardisation (CEN) has been supporting the development of STEP in the WG2 of its TC310, which is working in line with ISO. CEN/TC310 is responsible for the development of the Standards required by industry for the integration in Advanced Manufacturing Technologies (AMT) systems, such those required in the areas of Enterprise Modelling and System Architecture, Communication, Data, Information processing, Control equipment, Mechanical and System operational aspects. In other regions of the world, similar committees exist.

Moreover, in mid of 90s the Industrial Alliance for Interoperability (IAI) was created with the purpose of enabling software interoperability, providing a universal basis for process improvement and information sharing in the construction and facilities management industries (AEC/FM). Consequently, IAI developed the Industrial Foundation Classes (IAI/IFC) as an open standard model to allow software vendors to create interoperable applications via the IFC file format. Part 11 of STEP, and the ISO EXPRESS language (STEP-11) were also adopted by IFC to describe its models (IAI,2001).

At the same time, the Part Library Usage and Supply - PLUS project developed an exchange format for intelligent electronic catalogues, based on a common information model facilitating integration with third parties software. The results of this project contributed to the International Standard ISO13584: PLib (Parts Library). PLib intends to contribute a solution of electronic catalogue representation in proprietary formats, providing a tool for independent standard representation, supporting multi-representation and integration of different supplier catalogues using a consistent exchange and product modelling format, in this case based on ISO10303 STEP (PLib-1)(Pierra,G., 1997) (Fowler,J.) (Satub,G.,1998).

Other industries are also looking for interoperability between the APs covering their scope and those dealing for building and construction. The furniture industry is one example, which is seeking for interoperability between space definitions described by B&C data models, and furniture decoration projects described by furniture APs. The COFURN project, “Cooperation for consensus, standardisation and interoperability to support e-com services in the furniture industries sector” is supporting the development of STEP AP236 for furniture industry and the extensions in its architecture to have AP236 interoperable with space definition (COFURN, 2001)(Jardim-Gonçalves,R.,2001c).

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## 3 ARCHITECTURE FOR EXTENSION AND REUSE OF STANDARD DATA MODELS

Enabling software applications to adopt a standard for data exchange implies the development of skilled translators. These translators must realize the conceptual model using a standard modelling language, working as a bi-directional processor able to interpret and convert the data in the application's data format to the standard's one, making it possible to import and export the application's data (Jardim-Gonçalves,R.,1998) (Jardim-Gonçalves,R.,2001a) (Hars,A.,1998) (Putra,C.W.,1999).

Hence, these translators must be able to understand the software application's internal data structure and map it to the standard one following the rules for conversion defined by the standard. To implement it, several levels could be considered (figure 1).

The integration at the lowest level requires an intrinsic extension of the software application. This software extension should be strictly dedicated for the standard data model and developed from scratch. To implement at this level is consequently very costly. To make easier the adoption of standard models by the software applications, a Standard Data Access Interface (i.e., SDAI) could be used (ISO-22). This SDAI, defined mostly by the standards as a universal interface for data access in neutral format (ISO-21), is made available for use as a software library ready to be linked with the application. Assisted by a dedicated repository of data, this library is represented by a set of functions ready to handle and manage the data in neutral format.

Nevertheless, SDAI could be specified as a result from an early or late binding method. Early binding means to have the SDAI tailored to the standard data model adopted, providing static functionalities to access to the considered model. A late binding SDAI provides a general set of functions able to handle the data independently of the model, i.e. at the meta-dictionary level, supported by a meta-model dictionary loaded with the data model structure through a compilation process. Although the early binding approach is easier and faster, the late binding would provide immediate flexibility and extensibility of the model adopted without the requirement to extend the SDAI (Jardim-Gonçalves,R.,2000).

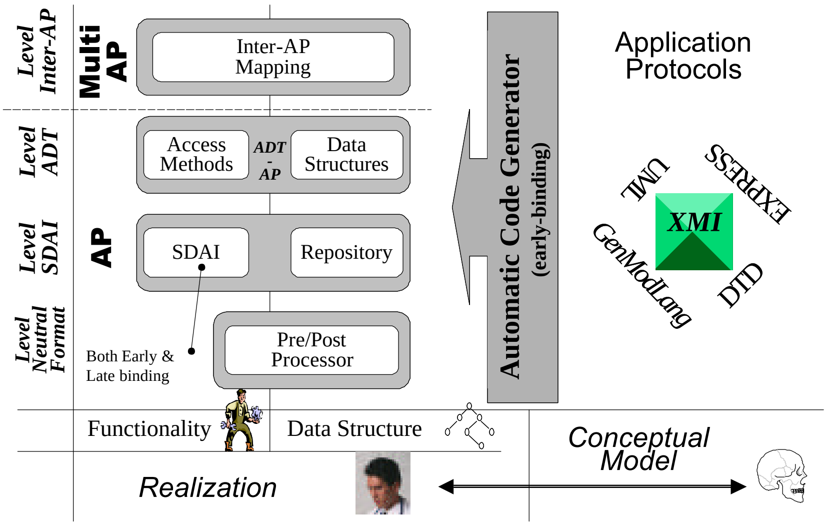


**Figure 1. Levels for AP-based integration**

In practice, to implement standards at SDAI level has been shown to require a long programming effort and deep knowledge not only about the model, but also about the standard implementation methods. It is this last requirement, i.e. requirement for deep knowledge about the standard implementation methods, which has prevented most of the software houses from adopting standards like STEP. Due to lack of resources, this is especially true of smaller companies and software houses, typical of those working for the building and construction industry.

At present it has been unanimously accepted that the levels of integration at SDAI and Neutral Format levels are of a level of detail too low to provide a convenient approach to software houses and modellers. To assist these companies adopting the standards for data exchange, an interface of higher level must be made available. These interfaces must be closer to the standard model data structure, realizing the conceptual description of the model as its Abstract Data Type (ADT). The ADT must offer, together with the data structure, a set of functionalities and methods that uniquely translate the objects instantiated from the ADT entities to the standard neutral format for export of data, and the reverse for import.

To avoid a case-by-case ADT development, automatic code generators could automatically create these libraries and pass the ADT to a programming language fitting the conceptual data model realized from the standard Application Protocol (Jardim-Gonçalves,R.,2001a) (Sousa,P.,1999a) (Sousa,P.1999b).

It should be noted that all the levels described and depicted in figure 1 exist when an application is adopting a standard. However, these levels, one or more, could be nested inside the application, or offered as integration platform library components. For instance, during the CIMTOFI and RoadRobot projects, models were developed in EXPRESS for integration purposes, and several translators implemented and linked with many of the applications running in the industrial environment at Neutral Format, SDAI and ADT levels (Jardim-Gonçalves, R.,1999b). This experience highlights the integration of the expert system shell - NExpert with the model described in EXPRESS. This facility allows the management and handling of knowledge based on the data model, and a very useful platform for the development of standard-based intelligent systems.

# 4 INTEGRATION AT MULTI-AP LEVEL

During the last few years, the standardisation communities have been quite active developing standard-based data models and application protocols for industry. The building and construction industry is one of those developing standards for data exchange, as are the IAI/IFC, STEP-AP225 or the emergent bcXML. Moreover, industries in the scope of their activities have the necessity to handle several applications in different domains.

Nevertheless, applications from different industrial domains very often need to interoperate, and the building and construction industry is perhaps one of those providing the best case to demonste that need. Examples are the cases of electrical installation in a building, furnishing a room, or the description of materials and components to be used for construction. However, these application protocols are nearly always developed using different modelling approaches and even, when defined using the same standard-based modelling methodology, mostly are not interoperable (Jardim-Gonçalves,R.,1999a) (West,M.,1998) (Jardim-Gonçalves,R.,1999c).

More recently, the need has been identified to have the possibility to extend, reuse and integrate dynamically application protocols that already exist. To enable this, extensions to the existent models at meta-model level, and support for knowledge is required. Using these model extensions on existent APs, they can be integrated in a new *implicit interoperable model*, i.e., a multi-AP, defined on the integration and mapping of selected views of the existent APs, driven by the AP's extension in knowledge and meta-model description.

This new approach is pioneering in terms of the definition of new "implicit" APs in a flexible and extensible way, where late or early binding mapping can be considered. It provides a methodology based on a new way to look at the development and use of APs at a higher level, where flexibility and interoperability are the essential issues, allowing softness in adoption and definition of dynamic protocols according to the domain and scope of the application. "Implicit" APs will be an important alternative for the static procedure defined by standards like STEP when developing APs, making possible a broader reuse of the already available APs.

## 4.1 Motivation to use XMI

When one application is willing to adopt a standard for data exchange, the first approach should be to look for those existent Application Protocols and select one, or part of it, to cover the application needs. If extensions on the selected AP are needed, those should be provided following the methodology described by the standard the selected AP is in.

However, very often the scope looked for is not always found in just one AP, but in one applicability domain covering more than one existent application protocols. In this case, rather than to develop a new AP, reusability is the best method to avoid "reinventing the wheel", saving all the work already done by the experts. The key issue in this scenario is to identify how to integrate these several APs, though most of them could be developed using heterogeneous representations. To achieve that, a multi-AP integration level should be considered. To integrate at this level, a common language must be adopted to enable a unique model language representation of the several APs.

Hence, the question is how to take the existent models described in languages like EXPRESS, reuse them and put them in the market in acceptable format like UML or any other? One immediate answer could be to develop model translators to the other format. But the core of the problem still persists, once models are described in different formats depending on the tools managing them, not existing a universal agreement for a neutral way to represent them.

Recently, the Object Management Group (OMG) (OMG,2001) released a proposal named XML Metadata Interchange (XMI) (XML,1998) (XMI,2001) intended to provide a general methodology for interchange of models and instances, at first to cover the OMG standards (e.g., MOF, UML, Corba)(Starzyk,D.,1999). XMI has been adopted by most of the popular toolkits available in the market as the reference for import/export of modelling information. To have mapping rules and translators from XMI to the languages used by the principal developed models, as are the case of EXPRESS or DTDs, would be an important step to assure reusability and acceptance of these models by the market (figure 1).

The key issue in this proposal is to assure the accurate translation in terms of semantics and rules between the several representations of the same model. The mapping between such languages usually is not direct, and therefore reliable translations are very often not simple to accomplish (Jardim-Gonçalves,R.,2001b).

## 4.2 Methodology to achieve ImAP: Integrated Multi-APs

A methodology to develop Integrated Multi-APs (ImAP) should start selecting the set of those Application Protocols that cover the applicability scope of the integrated environment considered.

For each selected AP, an extension must be applied to it in order to get an Extended for Interoperable AP (EIAP). This extension on each AP must define the description using the adopted meta-language of the AP at its Meta-model level, including the establishment of knowledge rules about the model semantics (figure 2).

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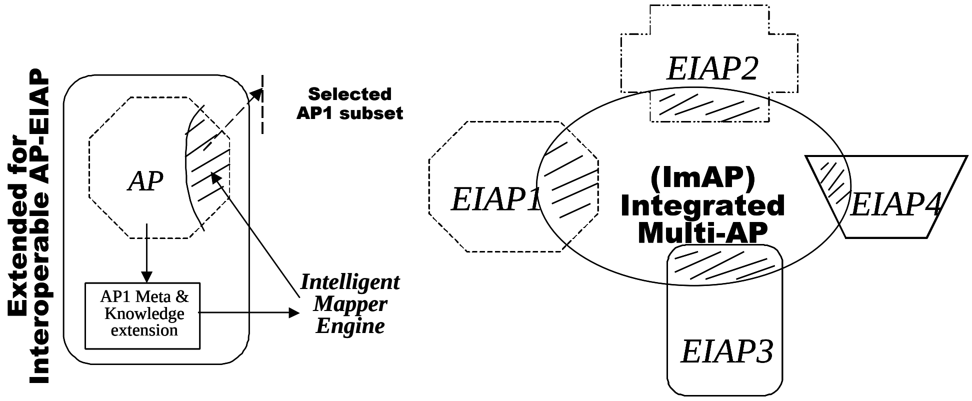


**Figure 2. ImAP using the EIAP concept**

Based on the requirements of the ImAP, and using the defined extensions for the AP at its meta level, a view corresponding to the required subset of the AP is defined. The Intelligent Mapper Engine (IME) assists this selection performing as the linker between all EIAPs to assemble the Integrated Multi-AP (ImAP) environment. The IME should support mapping between EIAPs at syntax, functionality, knowledge and semantic levels, with the transformation mapping functions of type AP1.a=f(AP2.a, AP2.b,…) which should be verified for all entities and attributes at stake, to achieve the ImAP environment using the EIAP concept (figure 3).

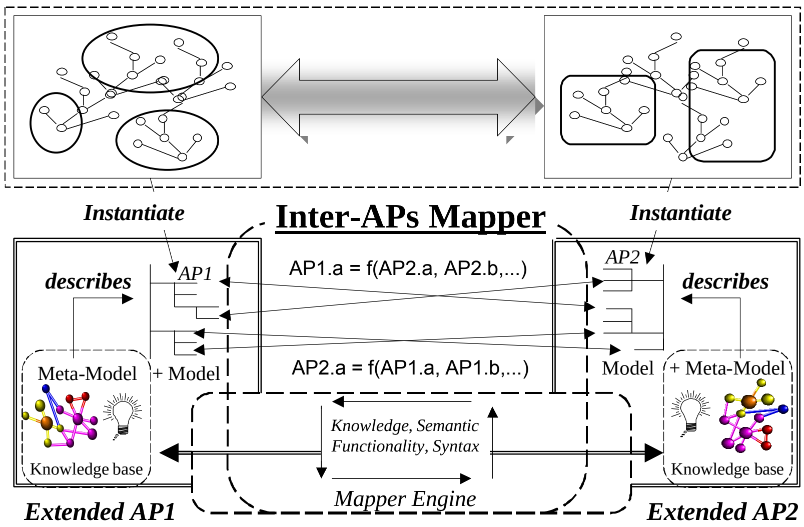


**Figure 3. Inter-APs Mapping architecture**

The ImAP can be developed and made available to the user following an early or late binding implementation. In the early binding approach, the IME will receive all the selected EIAP as an input, and will generate a new integrated AP to be later used by the applications accessing the integrated environment. In the early binding, the IME actuates before execution and hence "passively".

On the other hand, when adopting a late binding approach, the IME serves as the interface between the ImAP and the application, and it is the responsible for handling and managing the access to the data, i.e., on-line mapping and distribution among the EIAPs' repositories. In the late binding the IME acts at execution time "actively" on the subset of selected APs, generating implicitly the new Application Protocol.

## 5 INTELIGENT INTEGRATION WITH LIBRARIES OF MATERIALS

During the SUMMIT project, it was decided that the exchange of commercial and administrative data using EDIFACT standards would be complemented with the exchange of project management data, but using a more adequate standard for the exchange of product and process data.

SUMMIT focused on the developments on the Industry Foundation Classes (IFC) within the work of the International Alliance for Interoperability (IAI) (SUMMIT,1998), after an initial attempt to use developments in the Building Construction Core Model (BCCM) failed by the lack of in-depth work.

Despite the fact that IFC/IAI defines a clear model for the development and integration of scheduling and project management data with other components of the building process and product, there was an underlying philosophy of SUMMIT for systems integration of existing commercial applications and R&D developments, rather than an approach of major software developments or programming. Thus, the adopted approach is grounded on the concept that scheduling applications are linked to an IFC scheduling database (top-right of figure 4: *Manager:IAI/IFC based*).

The technological approach for the SUMMIT project considered the implementation of an integrated system combining IFC and EDIFACT information in a common communication infrastructure defined by an Internet-based inter-organizational workflow system, that co-ordinates the business and management information flows between project manager, contractors and suppliers in the various stages of building prefab wood houses.

Therefore, the need was clearly identified for an extension to the SUMMIT architecture to interoperate with standard-based catalogues of building materials and components described in PLib (Jardim-Gonçalves,R.,2000b). These libraries of Building Materials and Components described in a standard format could help in the design and scheduling tasks, since the process of search, selection and assignment of resources to tasks could be done in a normalized way, independently of the supplier. Moreover, if these standard libraries could include information about the intrinsic characteristics of the products in catalogue, e.g. robustness, durability, etc., it could enable intelligent searches to support the managing systems selecting those components and materials depending on proper criteria (top-left of figure 4: *Supplier: Plib-based*).

The key issue in this approach is related again to the interoperability problem. Most often, different materials and components suppliers describe their catalogues and libraries using different and incompatible data formats. This fact eliminates any immediate possibility for the managing systems to navigate and search in all available product catalogues in order to choose those products that best fits their requirements and needs. To contribute to the resolution of this difficulty, suppliers must be encouraged to adopt a standard format to represent their products and catalogues. The adoption of ISO13584-PLib standard can offer the basis for the required resources of this representation, where common dictionaries can be developed covering the different scopes of the catalogues contents (Sardet,E.,1997) (Jardim-Gonçalves,R.,1999d).

With the catalogues represented in standard format, an Intelligent Multiplexer Engine for Materials and Components (ImuxEMC) could be of extreme value supporting an independent supplier selection of the components and materials that best fit the requirements (bottom-centre of figure 4). The ImuxEMC receives the requests and selection criteria from the integrated platform as the input for the choice. Based on this input, ImuxEMC navigates through the available catalogues and returns back to its output the best-fit results.

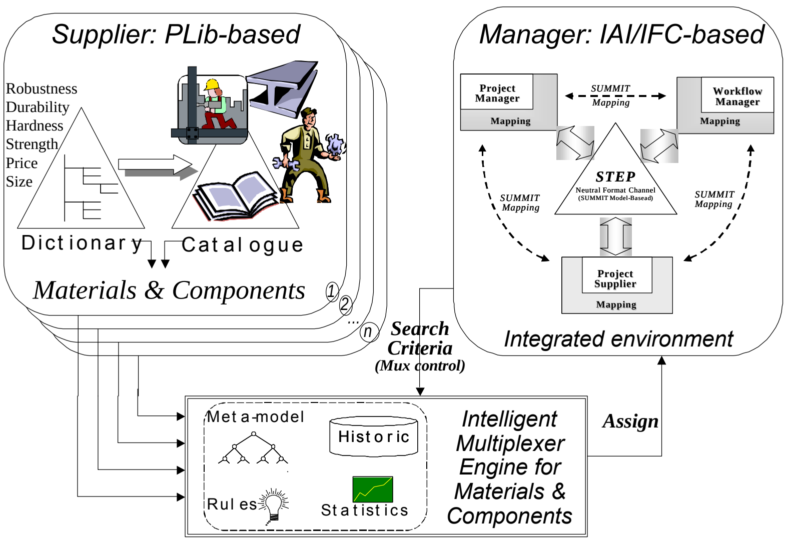


**Figure 4. Architecture for intelligent multi-level selection of materials and components**

The architecture of the ImuxEMC includes a rule-based engine assisted by a catalogue's dictionary meta-model. This engine is the core of the multiplexer's decision support system, assisted by rules describing the main features and functionalities of the components and materials in several contexts of usage applied in several case-based scenarios. The meta-model provides information about the semantics of the catalogue's dictionary classification, and together with the historic repository assisting the engine with records from past transactions, they provide the raw material for the learning of the system. Furthermore, statistics and mathematical indicators should be used as decision support indicators. Updated continuously, they will sustain more reliable and accurate results from the selection of the specified criteria.

The catalogues used as an input for ImuxEMC could be made available to the multiplexer engine in many ways, e.g., 1) via a common repository where the catalogues were previously downloaded; 2) each supplier can make available its catalogues via a storage media (e.g., CD) using the standard format; or 3) via web browser, enabling access to the servers where the catalogues are available in the standard format.

## 6 SUMMARY AND OUTLOOK

During the last decade many projects have been releasing significant results in the development of standard models and open interoperable platforms to support some of the major building and construction activities. IAI/IFC and ISO10303 STEP are two of the major examples of those standards most often used to model for this industry. These developments resulted from a considerable manpower effort involving numerous research and industrial experts. However, there is not yet an established agreement for the usage of a specific standard methodology for data representation.

The problem of interoperability is bigger when one application intends to cover more than one of the already existing APs, and would like to reuse parts of them to fulfil the requirements, i.e. looking for an “implicit” integrated multi-AP. To benefit from the efforts already made in the development of the existing APs, i.e. to avoid “reinventing the wheel”, the reusability of these existent data models must be considered. In order make such a procedure possible, each model should be translated to a common model language, and XMI as been accepted as a proficient one.

However, even with all APs described in a compatible language like XMI, to achieve the integrated multi-AP level requires a mapping procedure that should be supported by a rule-based system working on the meta-description and knowledge extensions of the selected APs. The integrated multi-AP emerges out to the user as a new “implicit AP”, reusing most of the work already done when initially developing the selected APs.

The authors believe that the future of interoperable open platform systems lie with the described “implicit” APs, where flexibility and reuse of existent models are applied to save the necessary huge effort to develop and implement a new protocol, including the reuse of test cases, conformance testing and interoperability checking procedures. On the other hand, “Semantic” mapping is a very hot research topic today, and still in its infancy. It deserves and will requires a lot of effort from the research community to produce practical solutions, rules and heuristics, and to offer the appropriate mechanisms to support a high level of interoperability.

Specifically for the building and construction industry, the integration of libraries of components and materials is a requirement when looking for “better buildings” in the advent of Internet and electronic data exchange. In this case, to adopt and push suppliers to describe their catalogues of products in a standard format, like PLib, is of major importance. Should this stage of standardisation be achieved, intelligent tools like the ImuxEMC presented in this paper could contribute seriously for “better data” during construction, resulting in improved quality of buildings and for those living in them.

## 7 ACKNOWLEDGMENTS

The work presented in this chapter results mainly from the bibliographic compilation, research and developments done by the authors contributing for the results of several European projects (CIMTOFI, RoadRobot, funStep, ECOS, FSIG, funStep AP, COFURN) and development of an Application Protocol for the furniture industry within ISO and CEN/ISSS (ISO 10303 AP236). The authors give thanks to the European Commission and to the PRODEP Portuguese program for the support and given funding.

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# Life-Cycle Cost Methods And Tools

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**Summary: Life cycle economics is the only way to control individual buildings and real estates over long terms. VTT, in co-operation with two consulting firms has developed a systematic way to apply life-cycle economics on different stages of building design. This model has been presented in this paper. It is based on investment costs modified to annual costs, yearly maintenance costs, renovation costs modified to annual costs as well as on an analysis of benefits like incomes, effects on quality, and other nonqualified factors.**

**Keywords: life cycle economics, building design**

## 1 ACKNOWLEDGEMENTS

This Life-Cycle Cost Estimation Methods (LCC) project has been carried out in co-operation with two Finnish consulting companies Granlund Olof Oy and Rapal Oy and VTT Building Technology research organisation. The first phase project was partly financed by the Technology Development Centre of Finland (TEKES) and ended in Dec.1997 and the second phase is still going on.

## 2 NEED FOR LCC METHODS AND TOOLS

The interviews with Finnish facility management staff and building designers have proven that there is lack of systematic methods and tools for design of buildings' life-cycle cost. However occupancy cost are usually the second largest cost in the business after labour cost. So better design of life-cycle cost seems to offer quite large saving potentials. The objective of this project is to develop life-cycle cost design methods and tools which are applicable both for building maintenance and design of new or renovation objects.

The phases of the life cycle are suggested to be as follows: acquisition, use and maintenance, renewal and adaptation and disposal.

The service life of buildings may also be divided in three life-times

- functional life-time (for example 20…25 years in case of residential buildings)
- technical life-time (for example 75…100 years)
- economic life-time (for example about 50 years)

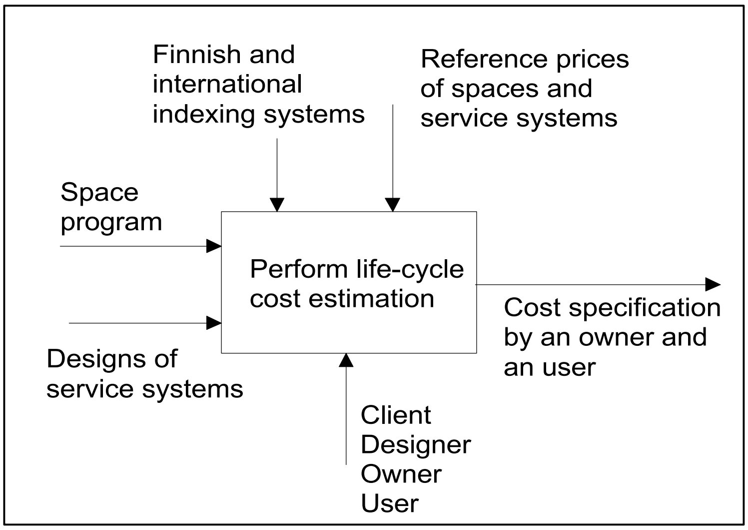


**Figure 1. Simplified activity model (IDEF-0) of the life-cycle cost estimation method.**

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Reference information about LCC models, abstarcts and files should be feeded to www, for which purpose are going on efforts in VTT (BENCHNET). The service life of products is dependent on it characters to be able to stay permanent as well as quality of production process.

On the whole, life-cycle management is an iterative process aiming at the selection of the most suitable and economic solution from many alternatives. Also consideration of buildings' environmental impacts should be part of the LCC design process.

Life-cycle costing can be applied from the very early need analysis design stages to the detailed design of building components. Life-cycle costing can roughly be divided in the following three design stages.

– analysis of functions, setting quality targets and level of energy consumption on the building level
– defining the building, its architecture, quality and functional targets on the system level
– building design on the component level

The estimation of profit on different levels is based on defining the annual demand of profit which covers yearly costs. Then the prototype uses as directive reference values when generating investing costs following rates of interest

– 10% in the period of ten years (directive meaning)
– 7% in the period of 25 years (suitable for example energy economic analysises)
– 4% in the total usage age of building (informatic meaning)

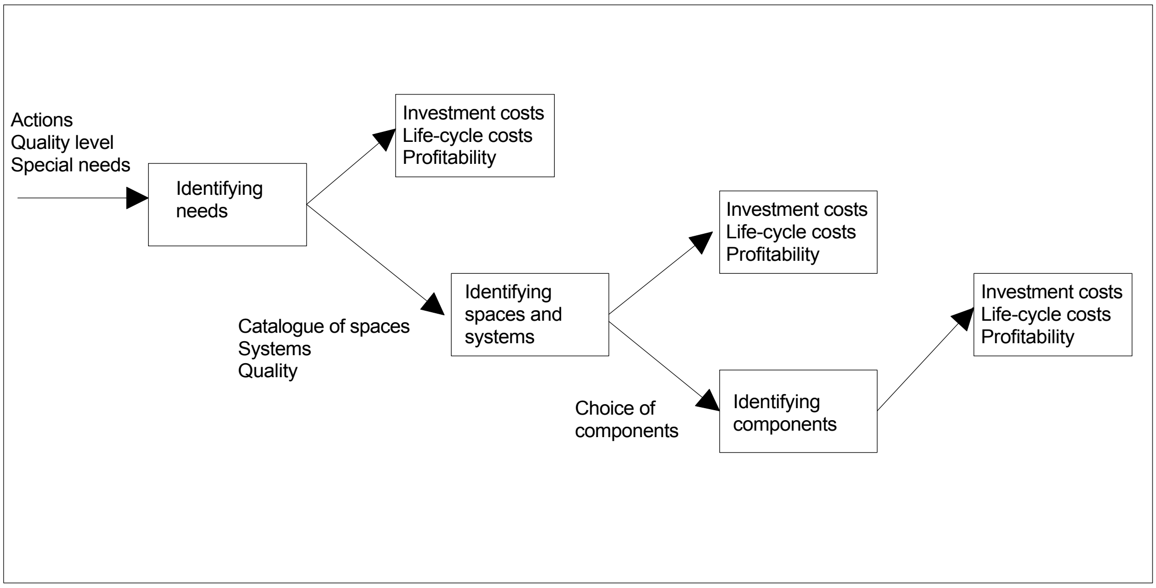


**Figure 2. Iterative life-cycle costing on different stages of building design.**

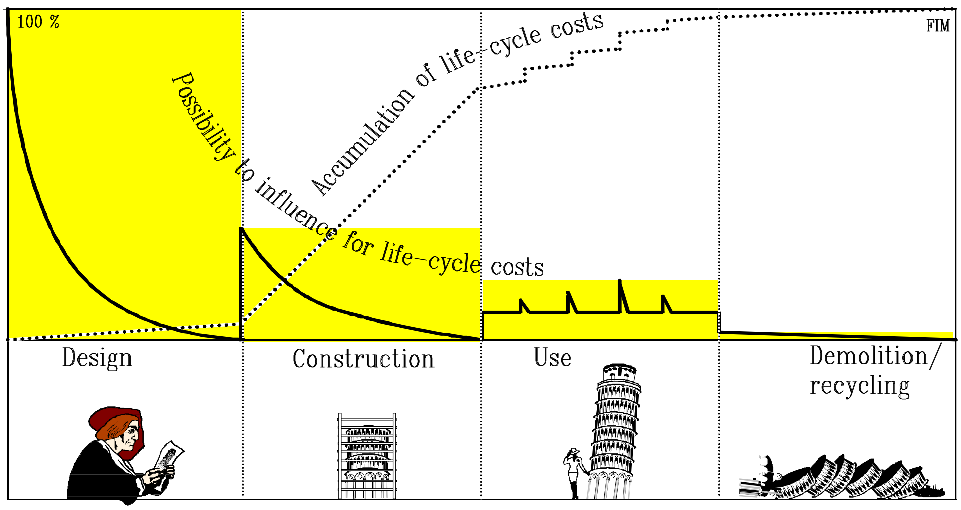


**Figure 3. Most part of life-cycle cost are fixed in early design stages.**

In this work the existing Finnish Space-Based Cost Estimation method is complemented especially with the maintenance cost factors of technical systems (HVAC and electrical) and the costs of these are divided for spaces. Occupancy costs will be reported from the viewpoints of a building owner and user. Also recommendation for a new indexing system for control of occupancy cost are under development.

## 3 LIFE-CYCLE COSTING MODELS AND TOOLS FOR BUILDING SERVICE SYSTEMS

During years 1993-1994 Granlund Oy developed models and prototype tools (HVAC) for the management of life-cycle cost of building service systems. By developing these models and tools further,it is possible to produce real software tools for different design stages and also devices to support decision-making in building maintenance. The central improvement areas are further development of price-list and consumption databases, integration of energy calculation software, definition of methods describing how to divide cost of service systems for spaces and development of a tool for electricity works management. The final objective is a software package including all the previous mentioned properties for comprehensive management of a building's life-cycle cost.

A standardised classification of maintenance cost has been published in Finland (Appendix 1, Figure 1). The main purpose of this classification is to make it easier to assign different facility management cost in a systematic way, draw up budgets, control maintenance costs, conduct accountings and understand the meaning of maintenance cost in connection with building planning. In this project we have interviewed Finnish real estate managers to clarify if they use this standard classification and what kind of further development needs they see in it. This project will recommend of new classification system for occupancy cost control. Life-cycle costs are reported according to this new classification and separated as cost of a building owner and a client.

## 4 LIFE-CYCLE MODEL OF FACILITY MANAGEMENT

Life-cycle cost consist of investment and maintenance (care and renovation) costs. In Tila-Suku facility management software developed by Rapal Oy investment and maintenance cost are defined per space unit.

Investment costs are allocated by space based on the Target Cost Method of the Finnish Building Cost Information System. Investment costs are transformed to annual cost with the annuity method. Then investment cost are comparable with the maintenance cost.

Maintenance costs are calculated with the help of the actual real estate maintenance cost information book published by the Building Economy Laboratory of Technical University of Helsinki. Consumption of heat and electricity, unit rates of cleaning works and renovations are allocated per space unit. Cost items which are not based on unit rates are allocated as space and client based (e.g. building managing, services, water, special equipment service, insurances, waste service) are divided for spaces by their areas.

The resulting target prices and maintenance costs are calculated. Generally information of actual cost are obtained in the form of total cost by cost items. In Tila-Suku the actual cost can be allocated for the spaces by calculated cost. Then for example rent or profit of each space can be estimated by investment and maintenance cost of this space.

In case of comparing concurring alternatives with each it may be reasonable to apply a manysided cost-effectiviness model (cost analysis, efefctivity analysis and alltogether) in which LCC calculations do usually have a leading role:

### 4.1 Cost analysis

- Input factors collection
- Cost calculations (product costs and building costs)
- Life-cycle costing
- Energy economical calculations

### 4.2 Effectivity analysis

- Price and value analysis, price flexibility, concurrence
- Use of time, Time of use
- buildability, integration with other technology, modifibility, terms of maintenance
- comfortability, Safety, Design (architecture etc.)
- Effects on environment
- marketing analysis
- effects on employment and skill

### 4.3 Alltogether

Scoring, alternative comparisons, Technological portfolio – optio, cash flow analysis, profit analysis, value analysis, sensitivity analysis.

## 5 RECOMMENDATIONS

The LCC-model and tool presented in this paper can be applied in various supporting tasks. The model can be used to develop computer-based tools for specific knowledge areas. Some kinds of tools are in use in different countries but much research work is still needed.

The recommendations are as follows

- we should develop a standard for LCC-model to be utilized in co-operation with different countries world-wide
- the model should cover spaces and technical systems
- the analysis should also include the analysis of the most important environmental effects on chosen technical systems
- the model must be independent of the information technology used in different countries and companies

## 6 APPENDIX 1.

The Finnish official classification of facility management cost

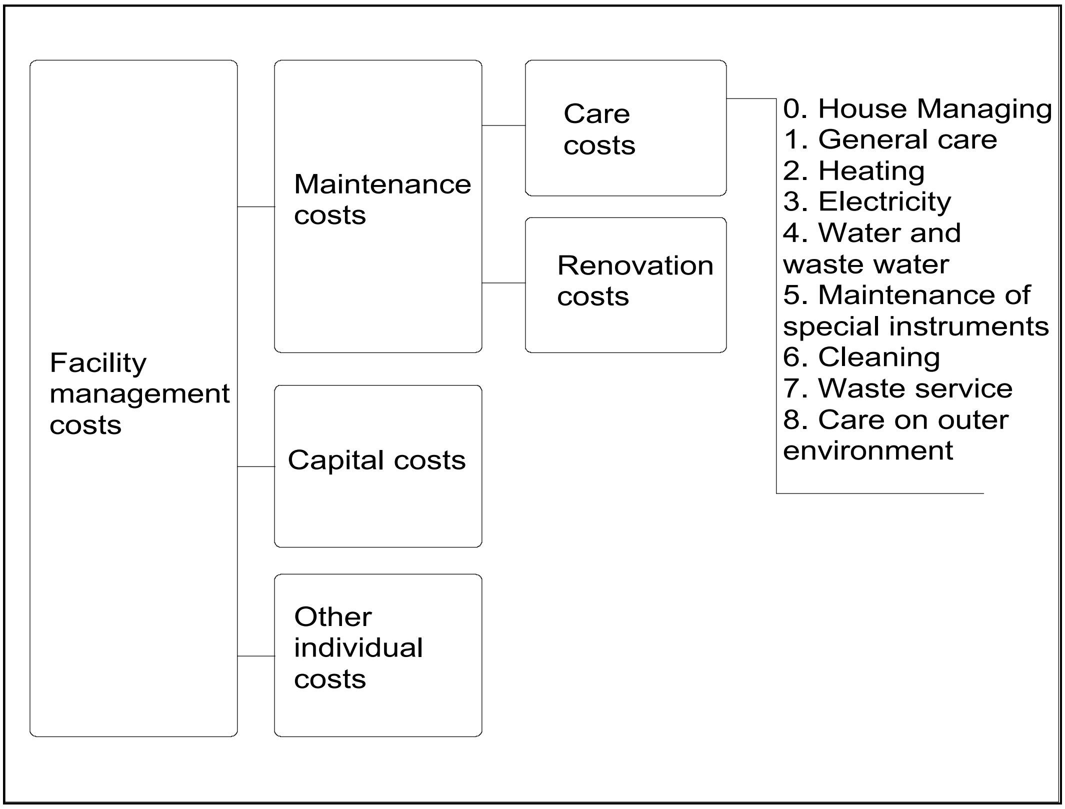


**Figure 1. The official classification of Finnish facility management cost.**

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# Stochastic Modelling Of The Facilities Management Costs In NHS Acute Care Hospital Buildings For Application In Whole Life Cycle Costing Methods

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**Summary: Whole Life Cycle Costing (WLCC) provides a valuable insight into the economic efficiency of constructed facilities. It has however, been criticised by practitioners and academics alike for producing forecasts of a high-risk nature, a consequence of the large number of assumptions inherent in the modelling process. Most WLCC systems use deterministic assumptions but a stochastic approach to modelling the assumptions will achieve a better representation of the likely costs. Monte Carlo and Latin Hypercube simulation can be used to create forecasts of the whole life costs but the accuracy of these are strongly correlated to the quality of the input probability distributions, hence the purpose of this paper is to present a methodology for the definition of probability distributions that best represent the facilities management costs in an acute care NHS hospital building**

**The data used in this research were obtained from National Health Service (NHS) Estates on the Facilities Management (FM) costs of over 450 acute care NHS Trusts in England and Wales. The data was analysed to obtain the parameters of the theoretical distributions that best describe the FM costs for a local NHS Acute Care teaching hospital building. It was found that the LogLogistic distribution represented the statistical majority of best fits. The distributions were then validated using various goodness-of-fit techniques. The result of this work might then be used as stochastic assumptions in the modelling of WLCC. The paper also discusses some issues of accuracy in distribution fitting, particularly the class interval rule and its effect on the quality of the results, and the selection of an appropriate goodness-of-fit test.**

**Keywords: Whole Life Cycle Costing, Probability Density Functions, Facilities Management Costs, National Health Service Acute Care, Hospital Buildings**

## 1 INTRODUCTION

WLCC is widely recognised amongst practitioners and academics as a valuable tool in assessing the economic efficiency of constructed facilities. It can be used as a means of comparing options and their associated costs and incomes over a period of time (CBPP 1998), or as a tool for assessing the long terms costs of ownership in existing buildings through stochastic modelling and key performance indicators (Kirkham and Boussabaine 2000a).

WLCC though is to a significant extent, dependent on assumptions about the future costs of operating and maintaining the building and its environment. It has been widely noted that concerns about using a WLCC approach are based mainly on the risky nature of the assumptions on which the forecasts are modelled (BRE 1999, Jovanovic 1999, Edwards and Bowen 1998). Whilst forecasting of future costs is to some extent an inexact science, this should not dissuade analysts and managers from attempting to apply WLCC principles (Woodward 1997).

As one of the leading providers of acute healthcare services in Europe, the United Kingdom NHS operates and manages a complex property portfolio of high occupancy buildings and establishments. Approximately £33.3bn of UK government spending is apportioned to the NHS, and some 22% of this accounts for estate management and capital investment (NHS Estates 1999a). The costs of maintaining and operating these buildings constitutes a significant proportion of total NHS expenditure on the estate, and in particular, the FM costs represent an integral part of this expenditure. As part of Audit

Commission guidance on NHS FM expenditure (Audit Commission 2000), a large proportion of the FM work is now outsourced to external private contractors (NHS Estates 1996, NHS Estates 1998, NHS Estates 1999b), and equipping professionals with the ability to monitor the cost effectiveness of these services should be an integral part of any NHS Trusts financial management programme.

NHS Estates collate annual data on FM costs from NHS trusts as part of the Trust Financial Proformas (TFP) returns (NHS Estates 1997), this gives stakeholders a snapshot of the variance of FM costs through the entire NHS estate. This is generally used to create simple benchmarks whereby NHS trusts can gauge the economic efficiency of their FM services against national averages.

However, the nature of this data poses problems for use in WLCC exercises. As the data is collated at trust level, it represents total spending on FM costs for all buildings within the Trust's estate, not for individual sites that make up the entire trust property portfolio. Most acute care trusts encompass several buildings within the estate and hence the data in its raw form cannot be used to monitor FM costs for individual buildings. To facilitate an accurate simulation of FM costs, the data must be transformed and analysed to reflect the cost of individual buildings within a Trust estate portfolio, not for the estate as whole. This paper proposes a methodology for addressing this issue.

## 2 RESEARCH METHODOLOGY

The purpose of this research is to present a methodology for the formulation of probability distributions of FM costs in a local NHS acute care trust building for the purposes a WLCC analysis. To perform this task, an original data sample of over 450 acute care NHS hospital Trusts' FM costs was used as the basis of the study. The first stage was to remove samples from the set that the contained FM data on non-hospital sites. Non-hospital sites, such as primary care buildings, clinics etc were removed because they do not reflect the true FM costs of acute care hospitals resulting in distorted hypothesis testing of the distribution fitting later on in the research. Once this had been performed, eliciting a set of observed data that had on aggregate, an approximately equal mean floor area to that of a typical ward block building in a university teaching hospital, then reduced the sample further. Similarly, building characteristics such as gross heated volume were used to further reduce the sample. After consultation with practitioners, the final data sample was reduced to 52 sets to eliminate data sets that had missing or erroneous data. All final data sets exhibited similar characteristics to that of the main ward block building used as the basis for the study (i.e. similar heated volume, occupancy and floor area).

The data sets were then statistically analysed for distribution fitting, using two software applications, ExpertFit™ and BestFit™, resulting in a distribution for each FM cost centre. Testing the distribution against 28 continuous probability distributions and the Chi-square goodness-of-fit test validated each distribution. The two packages were used to compare results and identify any ambiguity in first ranked fits. It was found that both packages yielded almost identical results.

## 3 FITTING PROBABILITY DENSITY FUNCTIONS TO FM COSTS

Most approaches for economic risk analysis use subjective probabilities to describe the uncertainty of input variables when historical data may not be available (Ranasinghe and Russell 1993, Perry and Hayes 1985, Bjornsson 1977). However, when historical cost data is available, the collation of this data and the subsequent modelling of real-world scenario's can give rise to several problems when trying to create valid probability distributions. A simple heuristic technique for assessing the validity of a distribution is to plot a histogram of the data and visually inspect the variance, kurtosis and skewness of the data over the range (Law 1998). This can give a basic suggestion as to which distribution (or family of distributions) best represents the data, but there are several factors, which must also be addressed before selecting a possible distribution.

Where is it possible to collect data on whole life costs on FM cost centres of interest, such data can be used to specify a distribution based on one of the following approaches: a trace driven simulation, an empirical distribution, or a theoretical distribution function (Maio et al 2000). If data is used to define an empirical distribution, the data is grouped to form a frequency histogram, and the resulting information is transferred to the simulation model. However, if the data set is used to fit a theoretical distribution using heuristics and goodness-of-fit techniques, it smoothes the irregularities that prevail and allows the possibility of sampling the extreme values of the distribution. This technique is regarded generally as the best method for performing simulations, and is used here for WLCC forecasts within the body of this research (Kirkham and Boussabaine 2000b).

## 4 PRE-DATA ANALYSIS: CLASS INTERVAL RULES

Class intervals, or "bins" are the ranges by which the data is grouped into on a histogram. The number and width of each class interval can have a significant bearing upon which distributions best fit the data being represented (when using the chi-square goodness-of-fit test). Some researchers (Montgomery and Ranger 1994) have suggested that the number of class intervals should fall in the region between five and twenty class intervals.

They suggested that the square root rule should be used to calculate the number of observations. Simply, taking the square root of the number of observations in the data set derives the number of class intervals. Sturges' Rule is reported on as another method of class interval selection (Maio et al 2000). Sturges' Rule states that, for $n$ observations, $X_i$ to be summarised in a frequency distribution, then the number of class intervals for the distribution should be calculated by:

$$K = \lfloor 1 + \log_2 n \rfloor \quad (1)$$

where K = number of bins and $n$ = number of observations, and

$$M = \frac{[X_{\max} - X_{\min}]}{K} \quad (2)$$

where M = width of class intervals, $X_{max}$ *and* $X_{min}$ = maximum and minimum values of observations in the data set.

## 5 GOODNESS OF FIT

Whichever method of calculating the number of class intervals in the distribution is used, the next stage is to fit a distribution to the data set. Although visual inspection can reveal which kind of distributions are most likely to represent the data, a statistical test should be performed to validate the choice of selected distribution. The Chi-square test is a formal comparison of the relationship between the observed data set and the theoretical distribution fitted. The Chi-square test though is highly correlated to the class interval rule chosen and as such, the method used to calculate the number of class intervals effects upon the Chi-square test results, particularly so in data sets where more than seventy observations are used. This has led to the conclusion that the Chi-square test is weakened by its dependence on the class interval rule. Notwithstanding, this test is widely used by construction researchers involved in fitting distributions to data sets. The chi-square statistic is defined as:

$$c^2 = \sum_{i=1}^{K} \frac{(N_i - E_i)^2}{E_i} \quad (3)$$

Where $K$ = number of bins, $N_i$ = the number of observed samples in the $i^{th}$ bin and $E_i$ = the expected number of samples in the $i^{th}$ bin

Inspection of Fig. 1 reveals that the class interval rule has no significant impact upon the accuracy of the distribution fitting procedure in this research. Sturges' rule and the square root rule converge at approximately 40 observations, and given the data set used in this study is 52, it can be concluded that either method will yield similar results, and not impact significantly upon the goodness of fit ranking procedure.

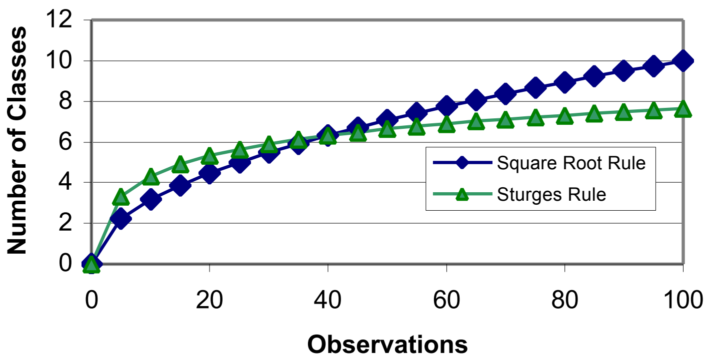


**Fig. 1: Comparison of Square root and Sturges' Rule for class interval calculation**

**Table 1: Results of distribution fitting for each cost centre**

| Cost centre | Class interval rule | Observed data | First ranked distribution using the Chi-square Goodness of fit test |
|---|---|---|---|
| Water cost | Square root | 52 | Inverse Gaussian |
| Sewerage cost | Square root | 52 | Logistic |
| Clinical waste disposal cost | Square root | 52 | LogLogistic |
| Domestic waste disposal cost | Square root | 52 | LogLogistic |
| Meal provision cost | Square root | 52 | LogLogistic |
| Laundry and linen services cost | Square root | 52 | LogLogistic |
| Porterage cost | Square root | 52 | LogLogistic |
| Patient transport cost | Square root | 52 | LogLogistic |
| Non-stock items cost | Square root | 52 | Erlang |
| Cleaning cost | Square root | 52 | Pareto |
| Patient records cost | Square root | 52 | LogLogistic |
| Sterile services cost | Square root | 52 | Rayleigh |
| Postage costs | Square root | 52 | LogLogistic |
| Capital charges cost | Square root | 52 | Logistic |
| Security cost | Square root | 52 | LogLogistic |
| Telecommunications cost | Square root | 52 | LogLogistic |

## 6 RESULTS

Table 1 shows the results of the fitting procedure. It can be observed that, when using the Chi-square goodness of fit test, the LogLogistic distribution was found to give first ranked fits for 62% of all distribution fits. However, in some cases where the LogLogistic distribution was not first ranked, the distribution was ranked sufficiently highly enough to consider its use in place of the first ranked distribution. It was therefore decided to assess whether the LogLogistic distribution could be used for the other cost centres where it was not the first ranked distribution (Water cost, sewage cost, non-stock items cost, cleaning cost and capital charges cost), through statistical justification. The purpose of this was to assess the validity of the hypothesis that the LogLogistic distribution is valid for all cost centres in FM services for acute care NHS hospitals.

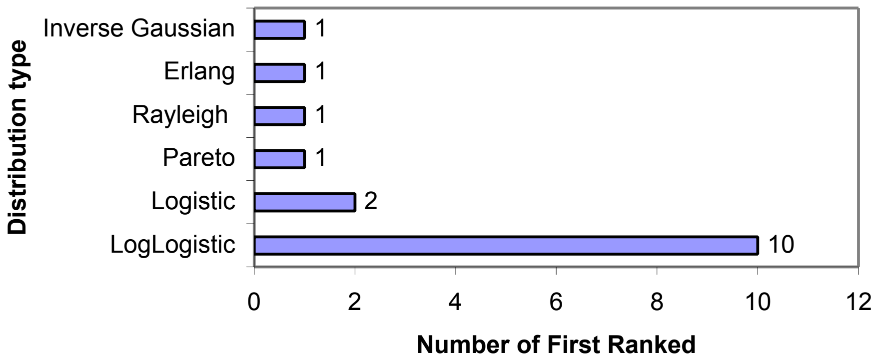


**Figure 2: Frequency comparison of first ranked distributions for all cost centres**

For each cost centre that did not have a first ranked LogLogistic distribution, it was decided to compare both the first ranked distribution and a hypothesised LogLogistic distribution on a pair-wise basis. To do this, the parameters of the first ranked distribution were obtained from the results discussed in Table 1. Then, for the relevant cost centres, a LogLogistic distribution was also fitted and again, the parameters were elicited. Simulation software was then used to generate 100 random samples for each distribution within the defined parameters. The purpose of this was to test whether the hypothesised LogLogistic distribution differed significantly from the first ranked distribution, based on a randomly generated sample of 100 cost observations.

## 7 VALIDATION OF THE HYPOTHESISED LOGLOGISTIC DISTRIBUTION

To test the validity of the hypothesised LogLogistic distribution, a simple hypothesis testing procedure was used. The distributions were compared using a two samples (assuming unequal variances) t-test. The t-test was used to examine a null and alternative hypothesis for each cost centre. If any of the two sets of distributions are similar, then the mean difference is expected to be around zero. But if the mean difference is much bigger than zero, then there will be a real difference between the distributions. Therefore, it is required that the null hypothesis of the mean difference being zero be tested. The following null hypothesis and alternative hypothesis were used for each cost centre.

1. $H_0$:$\mu_1$=0 the difference between the first ranked distribution and the hypothesised LogLogistic Distribution is small

2. $H_1$:$\mu_1 \neq 0$ the difference between the first ranked distribution and the hypothesised LogLogistic Distribution is large enough to suggest a real difference

Table 2 shows the statistics resulting from the t-test procedure. As a two-tailed test is used in this research, the P(T<=t) two-tail value determines the acceptance region of he null hypothesis. For P = 0.05, the null hypothesis is accepted and for P < 0.05 the alternative hypothesis is accepted. It was found that for three out of six cost centres (water cost: P(T<=t) two tail = 0.885, sterile services: P(T<=t) two tail = 0.708 and sewage cost: P(T<=t) two tail = 0.923) the LogLogistic distribution could be used in place of the first ranked distribution without any significant impact upon the accuracy of the results. The LogLogistic distribution could not be fitted to the capital charges cost centre because the distribution that best fitted this data is required to be of the unbounded continuous distribution type, not a non-negative distribution, such as the LogLogistic distribution. To reinforce the t-test results, the statistical differences between the first ranked distribution and the hypothesised Loglogistic distribution were calculated. To do this, a variety of statistical error measurements exist (Kirkham and Boussabaine 2000).

**Table 2: Results of 2-sample assuming unequal variances t-test**

| *Cost centre and distribution* | *Water cost centre* | | *Sterile services cost centre* | | *Sewage cost centre* | |
|---|---|---|---|---|---|---|
| *t-test descriptives* | *Inverse Gaussian* | *LogLogistic* | *Rayleigh* | *LogLogistic* | *Logistic* | LogLogistic |
| *Mean* | **83836.5073** | **83226.89968** | **357549.5966** | **368731.9895** | **82728.35819** | **83084.32622** |
| *Variance* | **884518111.1** | **906696376.7** | **45998796686** | **43436199931** | **689093296.9** | **685450348.2** |
| *Number of observations* | **100** | **100** | **100** | **100** | **100** | **100** |
| *Hypothesised mean difference* | | **0** | | **0** | | **0** |
| *Degrees of freedom* | | **198** | | **198** | | **198** |
| *t statistic* | | **0.14403783** | | **0.373921984** | | **0.096013373** |
| *P(T<=t) one tail* | | **0.442808543** | | **0.354431227** | | **0.46180353** |
| *t critical one tail* | | **1.652585979** | | **-** | | **-** |
| *P(T<=t) two tail* | | **0.885617086** | | **0.708862454** | | **0.923607059** |
| *t critical two tail* | | **1.972016435** | | **0.062786398** | | **0.062786398** |

| *Cost centre and distribution* | *Non-stock items cost centre* | | *Cleaning cost centre* | | *Capital charges cost centre* | |
|---|---|---|---|---|---|---|
| *t-test descriptives* | *Erlang* | *LogLogistic* | *Pareto* | *LogLogistic* | *Logistic* | *LogLogistic* |
| *Mean* | **16906073.69** | **12296638.21** | **2980826.112** | **1344829.55** | Invalid fit | Invalid fit |
| *Variance* | **3.0986E+14** | **9.16782E+13** | **4.5561E+13** | **1.95323E+11** | | |
| *Number of observations* | **100** | **100** | **100** | **100** | | |
| *Hypothesised mean difference* | | **0** | | **0** | | |
| *Degrees of freedom* | | **153** | | **100** | | |
| *t statistic* | | **-2.30030013** | | **-2.41855911** | | |
| *P(T<=t) one tail* | | **0.011391242** | | **0.008695281** | | |
| *t critical one tail* | | **-** | | **-** | | |
| *P(T<=t) two tail* | | **0.022782484** | | **0.017390561** | | |
| *t critical two tail* | | **0.062809704** | | **0.062864274** | | |

**Table 3: Error measurements statistics used in the validation process**

| *Cost centre* | *Error measurement statistics* | | | | | |
|---|---|---|---|---|---|---|
| | *Mean error* | *Mean absolute error* | *Sum of squared errors* | *Mean absolute percentage error* | *Theil's U statistic* | *McLaughlin's batting average* |
| *Water cost* | **16.58148** | **16.58148** | **26947420** | **0.0011223** | **0.002814** | **399.7185** |
| *Sterile services* | **64.26773** | **64.26773** | **40481480** | **0.007425** | **0.001171** | **399.8829** |
| *Sewage cost* | **20.97798** | **20.97798** | **43131820** | **0.015076** | **0.000663** | **399.9337** |
| *Non-stock items* | **-291541** | **291541** | **8.33E+14** | **0.785195** | **1.0** | **250.8448** |
| *Cleaning cost* | **-289573.9** | **289573.9** | **8.22E+14** | **11.61338** | **2.0** | **<100** |

Recent forecasting research (Kirkham and Boussabaine 1999, Boussabaine et al 1999) advocated the use of the Theils U statistic to test for differences between statistical models. Theils U calculates the difference by comparing changes in the observations of the first ranked distribution with changes in the hypothesised LogLogistic distribution. The U value is a coefficient value falling in the range between 0 and 2, where the difference is small as U tends to 0. The statistic is calculated using the following formula:

$$U = \frac{\sqrt{\dfrac{\sum_{t=1}^{n} e_t^2}{n}}}{\sqrt{\dfrac{\sum_{t=1}^{n} F_t^2}{n}} + \sqrt{\dfrac{\sum_{t=1}^{n} x_t^2}{n}}} \tag{4}$$

where $F$ = cost generated from hypothesised LogLogistic distribution, $x$ = cost generated from first ranked distribution, $n$ = number of observations and $e = x - F$

Table 3 gives the results of the tests. These statistical measurements supported the t-tests in that the water cost, sterile services and sewage cost all differed insignificantly from the hypothesised LogLogistic distribution, exhibiting U stats of 0.002, 0.001 and 0.00006 respectively, thus indicating almost identical fits.

## 8 DISCUSSION

Appendices 1, 2 and 3 show the final results of the fitting procedure post validation, providing the relative parameters and statistical descriptions. The two-stage validation process presented in this paper provided significant statistical justification for the use of the LogLogistic distribution in modelling the FM costs in acute care buildings in the NHS. After the hypothesis testing of non first-ranked LogLogistic distributions it was found that the distribution accounted for 81.25% of all first ranked distribution fits in all the FM cost centres. This was further supported by the analysis of both the variance of the skewness and kurtosis of each cost centre, which was relatively uniform, returning values of 0.515 and 8.503 respectively. Visual inspection of the distributions further supported this in that the costs were distributed principally around the lower end of the range of values, thus indicating a better fit for positively skewed distributions.

The results provide the analyst with a great deal of information about the costs of FM services when modelling the stochastic inputs into whole life cycle costing exercises. The use of empirical data in this research lends credibility to the assumption that the LogLogistic distribution is a suitable for representing FM costs. This is particularly useful in the analysis of whole life costs where empirical data to calculate the parameters of the distribution may not be available. Expert judgement can be used to form likely estimates of the parameters of the distribution, based upon the a priori assumption that the LogLogistic distribution best represents the FM costs.

## 9 CONCLUSION AND FUTURE WORK

The results of this research project support the overall hypothesis that the LogLogistic distribution is a powerful and accurate distribution for modelling all of the facilities management costs in acute care NHS buildings. Through a three-stage validation process, the accuracy of a hypothesised LogLogistic distribution was confirmed using historical facilities management cost data. These distributions can therefore be used as simulation inputs in whole life cycle costing exercises using Monte Carlo or Latin Hypercube simulation, for example, in forecasting the long term costs of FM services in acute care hospital buildings where historical data is unavailable to model assumptions. Generally, the triangular distribution is used in the absence of historical data but its weakness can have a significant impact upon the quality of simulation models. The knowledge gained from this research project provides evidence of the applicability a non-negative positively skewed distribution in this kind of cost modelling.

Using a case study, this paper has presented a methodology for developing PDF's of FM costs for a specific building. Similar studies should be conducted on distribution fitting of FM costs within predefined ranges of dependent variables for those such as heated volume, number of occupants, gross floor area etc. For example, from the original data set, the total range of gross floor areas could be identified and then divided into distinct equal intervals. For each interval, distribution fitting could then be employed throughout all the cost centres to assess whether a) the type of distribution differs for each cost centre as the floor area increases and b) the type of distribution is homogeneous throughout all cost centres and all gross floor area ranges. Identifying the type of PDF by ranges then provides practitioners with the ability to benchmark costs based on the characteristics of their own establishment, and offers the possibility to develop key performance indicators.

## 10 ACKNOWLEDGEMENTS

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### Appendix 1: Parameters and descriptive statistics for Logistic distribution

| Cost Centre | Parameters | | | Descriptive Statistics | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | γ | β | α | Mean | Mode | Median | Standard Deviation | Variance | Skewness | Kurtosis |
| Water cost | -546206.08 | 632584.4074 | 36.5148 | 87159 | 85430 | 86378 | 31508 | 992743234 | 0.188[est] | 3.6119[est] |
| Sewerage cost | -472819 | 558653.4 | 37.0882 | 86503 | 85022 | 85834 | 27393 | 7.5E+08 | 0.185[est] | 3.6102[est] |
| Clinical waste cost | -245702 | 335194.4 | 16.84673 | 91446 | 87139 | 89496 | 36554 | 1.34E+09 | 0.409[est] | 3.8147[est] |
| Domestic waste cost | -22600.4 | 58147.64 | 5.605916 | 38706 | 31924 | 35547 | 21215 | 4.5E+08 | 1.275[est] | 5.9991[est] |
| Meal provision cost | -1085570.3 | 2089396.36 | 7.946355 | 1059265 | 938343 | 1003826 | 505644 | 2.56E+11 | 0.882[est] | 4.7404[est] |
| Patient records cost | -1206764.9 | 1837627.637 | 8.8319 | 670194 | 584148 | 630863 | 395614 | 1.57E+11 | 0.790[est] | 4.5098[est] |
| Laundry and linen cost | -145131 | 573785.9 | 5.477928 | 461359 | 391254 | 428655 | 215516 | 4.64E+10 | 1.308[est] | 6.1206[est] |
| Porterage cost | -592866 | 1119743 | 8.910062 | 550419 | 498904 | 526877 | 238751 | 5.70E+10 | 0.783[est] | 4.4928[est] |
| Patient transport cost | -1834664 | 2337802 | 24.57662 | 509517 | 495406 | 503138 | 173573 | 3.01E+10 | 0.279[est] | 3.6778[est] |
| Sterile services cost | -1302790 | 1674678 | 13.08554 | 388085 | 352404 | 371888 | 237124 | 5.62E+10 | 0.528[est] | 3.9857[est] |
| Postal services cost | -655655 | 778694.6 | 38.28339 | 123915 | 121978 | 123040 | 36984 | 1.37E+10 | 0.179[est] | 3.6069[est] |
| Security cost | -28931.284 | 120182.4089 | 2.807842 | 120543 | 63245 | 91251 | 137088 | 1.88E+10 | 2.787[est] | 14.387[est] |
| Telecommunication cost | -1019122 | 1421720 | 15.31985 | 412621 | 390526 | 402607 | 170955 | 2.92E+10 | 0.450[est] | 3.8689[est] |

### Appendix 2: Parameters and descriptive statistics for Pareto distribution

| Cost centre | Parameters | | Descriptive Statistics | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | θ | a | Mean | Mode | Median | Standard Deviation | Variance | Skewness | Kurtosis |
| Cleaning cost | 0.573087 | 233221 | - | 233221 | 781709 | | | | |

### Appendix 3: Parameters and descriptive statistics for Erlang distribution

| Cost centre | Parameters | | | Descriptive Statistics | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | m | β | shift | Mean | Mode | Median | Standard Deviation | Variance | Skewness | Kurtosis |
| Non-stock items cost | 0.573 | 233221 | 8165 | 13718542 | 8165 | 9511474 | 13710377 | 1.88E+14 | 2 | 9 |

# A Risk Integrated Forecasting Model Of Electricity Cost In An Nhs Acute Care Hospital Building: An Application To Whole Life Cycle Costing

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**Summary**: **Through a case study, this paper reports on a research project to develop a risk integrated methodology for forecasting the cost of electricity in an NHS acute care hospital building. The paper is formed of two strands. Strand one presents a rationale for selecting an appropriate time series forecasting method and strand two looks at the implementation of probabilistic modelling of the forecasts generated in strand one. The results of the research revealed that the Holt-Winters multiplicative forecasting method produced the most reliable forecasts. The probabilistic modelling of the forecasts revealed that after a pair-wise comparison between data collected at the hospital used as the case study and data collected from NHS acute care trusts nationwide, the forecasts were most likely to belong to the weibull distribution. The results could then be used as inputs into a whole life cycle cost model or used a stand-alone forecasting technique for predicting future electricity costs for use in the NHS Trust Financial Proforma returns.**

**Keywords. time series forecasting, electricity costs, probabilistic modelling, hospital buildings, whole life cycle costing**

## 1 INTRODUCTION

The ultimate test of any forecast is whether or not it is capable of predicting future events accurately. Planners and decision makers have a wide choice of ways to forecast, ranging from purely intuitive or judgemental processes to highly structured and complex quantitative methods (Makridakis and Hibon 1979). Forecasting is a significant part of the whole life cycle cost exercise, and therefore, the approaches to selecting the correct method should be fully investigated. As a constituent part of a whole life cycle cost analysis, as defined in Kirkham and Boussabaine (2000), the analyst is required to produce future forecasts of the likely costs of electricity cost over a given period $n$. In other words, the basic problem being considered here is to forecast future values of an observed time series (in this case historical electricity cost data), denoted by $x_1$, $x_2$, …,$x_n$ (Chatfield 1997). The forecast made at time $n$ of $x_{n=k}$, where $k$ denotes the lead-time, will be denoted by $\hat{x}(n,k)$. At this point, it is pertinent also to distinguish between what constitutes a model and what constitutes a method in forecasting (Mead 2000). A model is an equation, or set of equations representing the stochastic nature of a time series forecast. A method is the combination of an estimating procedure and a model.

In establishing a forecasting procedure, it is vital to ascertain the objectives of the exercise. Not only must this be established clearly, but also a well-rounded knowledge of the background information needed to formulate the problem must be established. Chatfield (1997) discusses the importance of this iterative process; the needs to not only appreciate the theoretical aspects of selecting a model, but also the strategy that needs to be established in the modelling process. This paper is essentially formed of two distinct sections. The first section deals with the forecasting process and the models available in forecasting annual electricity costs in an NHS acute care hospital building. The forecasting methods discussed include naïve methods, exponential smoothing methods and extensions to exponential smoothing methods, namely Holt-Winters forecasting. The data generation process is described, and this includes details of the time series methods and simulation procedure. The summary statistics available to aid selection of the forecasting model are also presented. The second part of the paper discusses the risk associated with the forecasting process and proposes a novel method for modelling the forecasts (known as deterministic assumptions) as stochastic values. These stochastic values can then be used as inputs to a simulation model, which can be utilised for whole life cycle cost purposes.

## 2 FORECASTING METHODS

A plethora of forecasting methods currently exist and these were reported on in the M-Competitions, 1001 time series were used in the 1982 M-Competition (Makridakis et al 1982); 29 series from the 1993 M2-Competition; and 3003 series from the 1998 M3-Competition. Meade (2000) in his work on the selection of forecasting methods defined the methods into three distinct groups: the naïve methods, exponential smoothing methods and the Auto-Regressive Moving Average methods (ARMA). Naïve methods assume a minimal time series structure and comprise of three methods: 1) a long run average where an average of previous observations is used as a forecast such that $E(X_{T+L} \mid X_T,\ldots,X_1) = \hat{\Theta}_0$ for L = 1 where $\hat{\Theta}_0 = \frac{1}{T}\sum_{i=1}^{T} X_i$, 2) no change where the last known observation is used as a forecast such that $E(X_{T+L} \mid X_T,\ldots,X_1) = X_T$ for L = 1, and 3) a deterministic trend where an average of previous differences is used to estimate a global linear trend such that $E(X_{T+L} \mid X_T,\ldots,X_1) = X_T + L\cdot\hat{J}_0$ for L = 1 where $\hat{J}_0 = \frac{1}{T-1}\sum_{i=2}^{T}(X_i - X_{i-1})$.

The second group of methods are those that use exponential smoothing. Exponential smoothing methods are discussed in Makridakis et al (1993) who identify six variations of exponential smoothing: linear, linear (Holt's two parameter method), Pegels' classification, quadratic (Brown's one-parameter method), seasonal and single. These six methods can be further subdivided into seasonal and non-seasonal methods. With respect to seasonality methods, the Holt-Winters Multiplicative and Holt Winters Additive methods were reported on in Chatfield (1978), as techniques that were able to better represent trend and seasonality in data, such as that what would be expected of electricity consumption data. The final group, the ARMA methods are sophisticated techniques and as these techniques are explained in Fildes et al (1998), Box and Jenkins (1970) and Makridakis et al (1993).

## 3 SELECTING AN APPROPRIATE FORECASTING METHOD

It is very difficult to specify a rationale for selecting an appropriate forecasting technique. This is evident in Collopy and Armstrong (1992) who present 99 different rules to facilitate the selection of annual data from 4 forecasting methods. For data which exhibits trend and seasonality such as electricity cost data, then a simple seasonal forecasting technique is best applied, indeed in Fildes and Makridakis (1995) evidence is presented of the out-performance of complicated forecasting techniques by simple seasonal methods. However, it is clear that some kind of integration between statistical measurement and expert inference is preferable, particularly in Armstrong and Collopy (1998) who concluded that the most reliable forecasting techniques were identified as the ones that used a combination of statistical testing and expert domain knowledge. The researchers also concluded that time series data which exhibited trend and seasonality was more likely to yield an accurate forecast than other data. This would validate the discussion in Chatfield (1992), which recommends the use of the Holt Winters forecasting technique for forecasting seasonal data. However, is forecast accuracy the sole decision criterion in selecting the forecasting model? Studies by Yokum and Armstrong (1995) and Makridakis and Hibon (1979) challenged the theory that accuracy was not the only factor in selecting a forecasting model. Interpretation, cost/time and ease of use all figured highly in decision criteria as well. Software packages that are currently available provide the analyst with the ability to use several different techniques simultaneously so that the relative accuracies can be compared statistically. Kirkham et al (1999) and Boussabaine, Kirkham and Grew (1999a, 1999b) in their work on forecasting energy costs in sport centres advocated the use of Theil's U statistic as an appropriate form of differentiating between different methods. The software packages available tend to rank forecasting methods based upon the Mean Absolute Percentage Error (MAPE), as was the case in the M-Competition, although a variation on this, the Unbiased Absolute Percentage error was proposed in Collopy and Armstrong (2000). It is widely acknowledged however that if the analyst is making an a priori assumption as to which forecasting technique is best suited to the task, then the analysts' skill and intuition will be significant in terms of the success of the forecasting method.

## 4 INTEGRATING RISK INTO THE FORECASTING PROCESS

Research aimed at modelling the uncertainty in the time series forecasting process has not been as well documented. Forecasting research tends to concentrate on the point forecasts produced but what may be more appropriate is a Prediction Interval (PI)(Chatfield 1997), which is an interval forecast associated with a specific probability. It was noted that PI's are generally too narrow and too dependent upon past data, that is assuming that the future is like the past. This argument though to some extent is weak in that most forecasting techniques place a heavy emphasis on the past as a basis for predicting the future. Armstrong and Farley (1969) integrated a Markov Chain into the forecasting process although they found in their results that the integration of this stochastic technique had little impact upon the forecasting accuracy. Markov Chain's have been used also in other forecasting applications (Wirahadikusumah et al 1999, Cesare et al 1992 and Scherer and Glagola 1994) but these were not incorporated into time series forecasting and thus not discussed in this work

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## 5 RESEARCH METHODOLOGY

The aim of this paper is to demonstrate a methodology for forecasting the costs of electricity in an NHS acute care teaching hospital building in the UK. The results of this could then be used as stochastic inputs to a whole life cycle cost model as discussed in Kirkham and Boussabaine (2000). To facilitate this, the research forms two distinct strands. Strand one deals with the methods of selecting and implementing an appropriate time series forecasting method of total electricity cost in the hospital building. Data on electricity consumption from a large university teaching hospital building was obtained and subsequently, a four-year forecast is produced. Strand two deals with the methods and implementation of a strategy to model the forecasts produced as probabilistic distributions to model the uncertainty of each point forecast produced. The outcomes of this research address two key requirements: 1) provide an novel and innovative method of modelling electricity costs in the hospital building as stochastic WLCC inputs and 2) provide a methodology for NHS estates managers to forecast future electricity costs effectively in line with the requirements set out by NHS estates and the annual Trust Financial Proforma's (TFP) returns (see NHS Estates (1997) for more detail on TFP exercise).

### 5.1 Strand one: Time series forecasting of electricity cost

For the purposes of this paper, it was decided to set the forecast period for 4 years (the number of years required to be forecasted by NHS Estates). However, due to data availability and time lags in receiving historical cost data, the basis of the forecasting procedure would be on historical data over the period April 1997-Dec 1999. Therefore the period of forecasting would be up until the Year 2003. This does have the advantage though of giving the researchers the ability to validate the quality of the forecasts at the end of this year when the cost data for 2000/01 is made available.

The hospital building used in this study is the main ward block building of an NHS Trust Teaching hospital in the north west of England. The building is approximately 24 years old and is constructed of a concrete frame and pre-cast concrete cladding with single glazed windows at an approximate ratio of 1:3. The gross floor area of the hospital building is approximately 83,1366 metres squared and has an approximate heated volume of 2,029.00 metres cubed. Three electricity supply sub-stations service the main ward block building. Substation 3, substation 4 and substations 3 and 4 High Voltage Supplies, which source the supply from a combination of in-house produced electricity and external supply for a local power company.he first task was to elicit the monthly meter readings from each substation, in kWh. To convert the readings into unit cost a moving average was used as to the unit cost of electricity each month varied.

**Table 1: Forecasting results for all substations**

| Substation 3 Methods | Rank | Root Mean Square Error (RMSE) | Mean Absolute Deviation (MAD) | Mean Absolute Percentage Error | Theil's U Statistic | Alpha | Beta | Gamma |
|---|---|---|---|---|---|---|---|---|
| Double Exponential Smoothing | 7 | 3200.6 | 2747.2 | 18.958 | 0.681 | 0.168 | 0.999 | |
| Double Moving Average | 8 | 3317.8 | 2927.2 | 20.569 | 0.523 | | | |
| Holt-Winters' Additive | 3 | 2298.6 | 1665.1 | 11.595 | 0.481 | 0.041 | 0.999 | 0.001 |
| **Holt-Winters' Multiplicative** | **1** | **2285.8** | **1559.6** | **10.69** | **0.489** | **0.001** | **0.001** | **0.001** |
| Seasonal Additive | 4 | 2408 | 1772.6 | 11.86 | 0.536 | 0.107 | | 0.001 |
| Seasonal Multiplicative | 2 | 2298.3 | 1633.1 | 10.633 | 0.543 | 0.012 | | 0.001 |
| Single Exponential Smoothing | 5 | 2676.4 | 2312.8 | 15.178 | 0.593 | 0.059 | | |
| Single Moving Average | 6 | 2798.3 | 2390.4 | 15.992 | 0.558 | | | |
| **Substation 4 Methods** | **Rank** | **Root Mean Square Error (RMSE)** | **Mean Absolute Deviation (MAD)** | **Mean Absolute Percentage Error** | **Theil's U Statistic** | **Alpha** | **Beta** | **Gamma** |
| Double Exponential Smoothing | 7 | 2604.7 | 2142 | 17.178 | 0.636 | 0.144 | 0.999 | |
| Double Moving Average | 8 | 2885.7 | 2512.2 | 20.402 | 0.544 | | | |
| Holt-Winters' Additive | 3 | 1994.4 | 1586.7 | 12.611 | 0.515 | 0.046 | 0.999 | 0.001 |
| **Holt-Winters' Multiplicative** | **1** | **1892.8** | **1302.6** | **10.283** | **0.492** | **0.001** | **0.001** | **0.001** |
| Seasonal Additive | 4 | 2059 | 1589.6 | 12.337 | 0.543 | 0.115 | | 0.001 |
| Seasonal Multiplicative | 2 | 1924.2 | 1387.5 | 10.512 | 0.534 | 0.001 | | 0.001 |
| Single Exponential Smoothing | 5 | 2386.4 | 1997.9 | 15.175 | 0.613 | 0.099 | | |
| Single Moving Average | 6 | 2435.1 | 2074.2 | 16.211 | 0.56 | | | |
| **Substations 3 and 4 HV Methods** | **Rank** | **Root Mean Square Error (RMSE)** | **Mean Absolute Deviation (MAD)** | **Mean Absolute Percentage Error** | **Theil's U Statistic** | **Alpha** | **Beta** | **Gamma** |
| Double Exponential Smoothing | 8 | 8980 | 7427.7 | 30.833 | 0.903 | 0.486 | 0.398 | |
| Double Moving Average | 7 | 8228.7 | 7037.8 | 27.365 | 0.812 | | | |
| Holt-Winters' Additive | 2 | 6096.2 | 4468.6 | 18.633 | 0.637 | 0.04 | 0.999 | 0.001 |
| **Holt-Winters' Multiplicative** | **3** | **6273.4** | **4017.2** | **16.591** | **0.613** | **0.001** | **0.257** | **0.001** |
| Seasonal Additive | 4 | 6333.7 | 4925 | 19.972 | 0.692 | 0.197 | | 0.001 |
| Seasonal Multiplicative | 1 | 5981.7 | 4348.4 | 17.082 | 0.662 | 0.067 | | 0.001 |
| Single Exponential Smoothing | 6 | 7266.2 | 5217 | 19.476 | 0.854 | 0.208 | | |
| Single Moving Average | 5 | 6781.8 | 5120.4 | 20.61 | 0.788 | | | |

The cost data was then exported into a forecasting software package, CB predictor, which was used to generate forecasts for the period Jan 2000 to Dec 2003. CB predictor uses four components: level, trend, seasonality and error to analyse the historical data and then produce the specified forecast (Decisioneering 2000). The software has the power to generate up to eight different forecasts using for non-seasonal and four seasonal forecasting methods. The non-seasonal methods available are single moving average, double moving average, single exponential smoothing and Holt's double exponential smoothing. The four seasonal methods available are seasonal additive smoothing, seasonal multiplicative smoothing, Holt-Winters' additive seasonal smoothing and Holt-Winters' multiplicative smoothing. Data pre-analysis revealed that it would be likely that the seasonal methods would most accurately forecast the future costs but a comparison of all eight methods for each substation is presented in Table 1 which also gives the goodness-of-fit parameters of each forecasting method. The results of the forecasting procedure were review by an independent energy management specialist at the NHS trust and after a consultation process it was found that the forecasts for substations 3 and 4 using the Holt-Winters Multiplicative method were satisfactory. However, for the substation 3 and 4 high voltage, the expert cast doubt over the validity of the forecasts produced by the Holt Winters Additive method so a lower ranked method (Holt-Winters Multiplicative) was selected in preference. Fig. 1 shows a graphical representation of the forecasting results, in this case the forecast for the No.3 substation is shown

The monthly point forecasts were then aggregated to form an annual cost for each substation, for each year from 2000 to 2003. Finally, to calculate the total cost of electricity supply to the hospital building, all three substations were aggregated together for each year.

### 5.2 Strand two: Stochastic modelling of annual forecasts.

The next stage of the process is to use the results generated in the first strand and transform the forecasts into probabilistic values, which can then subsequently be used as stochastic inputs into a whole life cycle cost model. However, in modelling the forecasts as probabilistic values, knowledge is required about the conditions surrounding the variables and the most likely probability distribution that the forecasts belong to. This can prove difficult but the selection of an appropriate probability distribution is very important as it can have a significant effect upon any simulation work that is carried out in the future to calculate the whole life cycle cost.

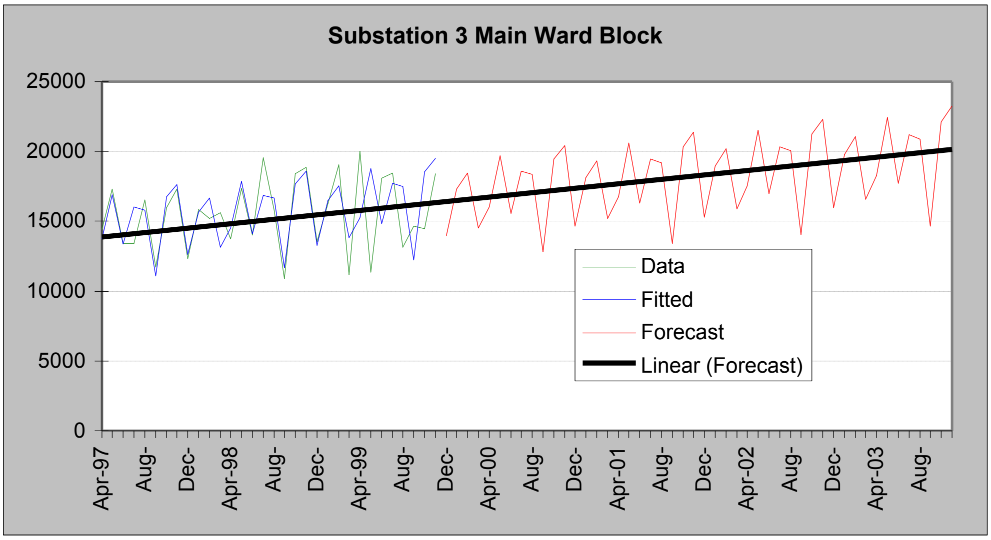


**Fig 1: Fit and Forecast for No. 3 Substation**

Therefore, to establish the most likely distribution that the forecasts would come from, a database of historical energy costs for over 450 NHS acute care trusts in England and Wales was analysed, for the periods 1996/97, 1997/98 and 1998/99. The first stage was to remove samples from the sets that contained energy cost data on non-hospital sites. This was performed because the data was collated at trust level i.e. for all buildings within a trust's estate, not for individual buildings. Non-hospital sites were removed because they do not reflect the true energy costs of acute care hospitals and would thus distort distribution fitting later on in the research. Once this had been performed, eliciting a set of observed data that had on aggregate, an approximately equal mean floor area to that of the ward block building in the university teaching hospital, then reduced the sample further. Similarly, building characteristics such as gross heated volume were used to further reduce the sample. After consultation with practitioners, the final data sample was reduced to 52 sets to eliminate data sets that had missing or erroneous data. All final data sets exhibited similar characteristics to that of the main ward block building used as the basis for the study.

A probability distribution fitting programme was then used to transform the data from each year into probability density functions. The purpose of this being to establish any similarities between each years cost data and also to establish any particular distribution, or family of distributions that best represented the cost data. The results of this exercise are shown in Fig 4 where the distributions and the parameters of the electricity cost data for each year's data set are presented. It was found that in this case, for acute care hospital buildings with the same physical characteristics to that of the hospital building used in this study, the Weibull distribution was ranked first in terms of goodness-of-fit for the periods 1996/97 and 1998/99 and ranked second in the period 1997/98, based upon the Andersen-Darling (A-D) goodness-of-fit test. The A-D test was used in this work in preference to the other goodness-of-fit tests discussed in Maio et al (2000) because of its power to detect any discrepencies in the tails of the distributions. Also, the class interval rule that is fundamental aspect of the Ch-square goodness-of-fit test does not apply to the A-D test thus giving the results greater credibility.

Equipped with the knowledge that the annual costs of electricity in this hospital were likely to come from the Weibull distribution, the monthly point forecasts generated in strand one of this paper when then used in a Monte Carlo simulation. The purpose of this was to generate a distribution of the annual forecasted cost for the years 2000 to 2003. Two thousand iterations were used in the simulation process and the results of this process are presented in Table 2. The Table shows the A-D ranking of the best fitting distribution and the relative position of the Weibull distribution. The A-D test was used to assess the validity of the Weibull distribution in preference to the first ranked distribution and it was found that for all 4 years, the Weibull distribution was accepted by the A-D test and that the error in the weibull model mean relative to the data sample mean (d) fell in the region of 0.01% = d = 0.02% for all forecasts.

**Table 2: Results of Monte Carlo Simulation and comparison with Weibull distribution**

| Forecast year | First ranked distribution | Andersen – Darling test | Weibull distribution ranking | Andersen – Darling test | Error in model mean relative to sample mean (%) | Accept/reject weibull distribution |
|---|---|---|---|---|---|---|
| 2000 | Random walk | 0.28220 | 4 of 22 | 0.83723 | 0.01 | Accept |
| 2001 | Random walk | 0.74274 | 5 of 22 | 2.23663 | 0.02 | Accept |
| 2002 | Johnson SB | 0.33149 | 2 of 23 | 0.32890 | 0.01 | Accept |
| 2003 | Lognormal | 0.51287 | 4 of 22 | 1.12314 | 0.01 | Accept |

Figs 2, 3, 4 and 5 show the final fit of the weibull distribution for each annual forecast and a probability-probability (P-P) plot. For the P-P plot let $X[1]$, $X[2]$, ..., $X[n]$ be the sample values arranged into increasing order. Therefore, the P-P plot is a graph of a model distribution function evaluated at $X[i]$ versus the sample distribution function evaluated at $X[i]$, *for* $i = 1, 2, ..., n$. If the model distribution function is the same as the true underlying distribution function of the data and n is "large," then the model and sample distribution functions will be close together and the P-P plot will be approximately linear with an intercept of 0 and a slope of 1. Even if the model is the correct distribution, there will be departures from linearity for small to moderate sample sizes. The P-P plot is designed to amplify differences that exist between the "middle" of the model distribution function and the "middle" of the sample distribution function.

## 6 DISCUSSION

The final distributions presented in Figs 2, 3, 4 and 5 reveal the quality of the goodness-of-fit of the weibull distribution for the yearly forecasts, which validates the early work where it was found that for similar NHS acute care hospital buildings nationwide, the underlying distribution of costs of electricity were likely to belong ot the weibull distribution. The weibull distribution is very flexible as it can take a variety of distribution shapes, for example the weibull density function (0,1,3) is very similar to a lognormal distribution (positively skewed) and the weibull density function (0,1,1) is very similar to the exponential function.

The results show that for all the forecast years, the distribution is positively skewed for each forecast, indicating that costs were likely to fall within the lower end of the range. The variance and kurtosis for all forecasts were relatively constant; the lowest variance being 2.42005 e 8 for year 2000 and achieving a maximum of 3.50989 e 8 in the Year 2003 and the kurtosis varying between 2.82379 in Year 2002 and 3.26848 in Year 2003. The P-P plots show conclusively the goodness of fit of the Weibull distribution in preference to the first ranked distribution selected by the software. One the P-P plots a comparison is made of the first ranked and weibull distributions, where the closer the fit of both plots, the better the fit of the weibull distribution. For all years, the weibull distribution fitted well in comparison with the first ranked distribution, and the relative error for the weibull distribution fell in the region 0.03% to 0.08%, again indicating a very close fit.

Generally, the results provide a method whereby future forecasts of electricity cosumption can now be quantified as stochastic forecasts, using integration, the analyst can now prescribe a determined level of confidence associated with the forecasts. This addresses the main problems associated with the whole life costing methods, that is the risky nature associated with long term future forecasts. This method provides a way to forecast accurately, and define the uncertainty assocaited with this kind of modelling

## 7 CONCLUSIONS

This paper reviewed the current methods used in time series forecasting and discussed the relative merits of the techniques. It was found that all though statistical measurements are generally used to assess the accuracy of the forecast, expert judgement can be used in conjunction with the tests to improve accuracy. It was found in this research that the Holt-Winters Multiplicative forecasting method yielded the best results and this was validated by expert opinion and statistical error

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measurement. The forecasts were the treated as stochastic assumptions and the annual cost forecast for four consecutive years was simulated using Monte Carlo methods. It was found that in a data sample of NHS acute care hospital buildings throughout the UK, electricity costs were weibull distributed, and after comparing this with the first ranked distribution for each annual forecast, it was found that the weibull distribution also had a high level of accuracy in terms of goodness of fit. These results could then be used in a whole life cycle cost model or as a stand-alone methods for forecasting electricity cost with risk.

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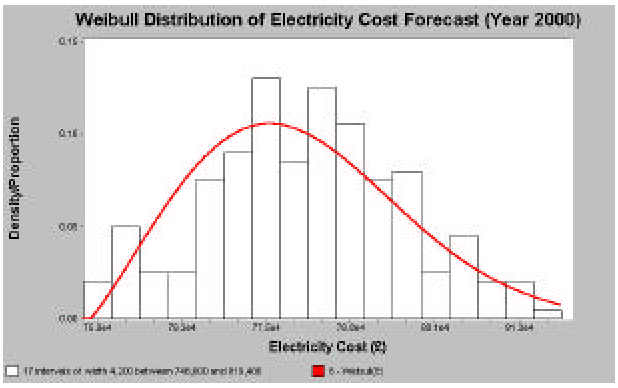
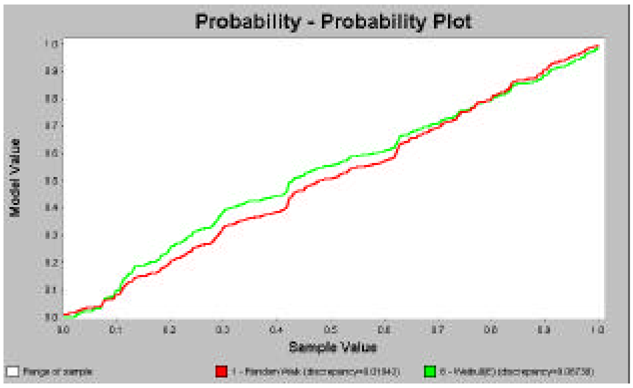
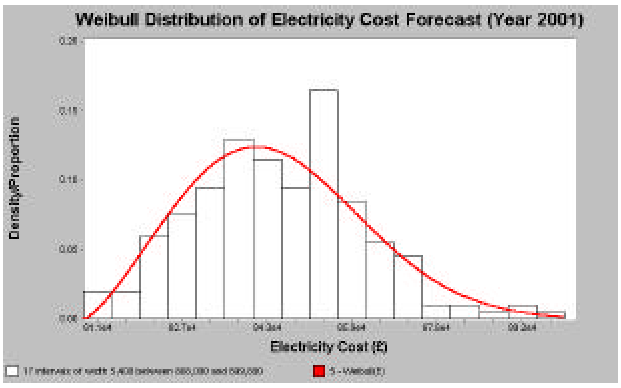
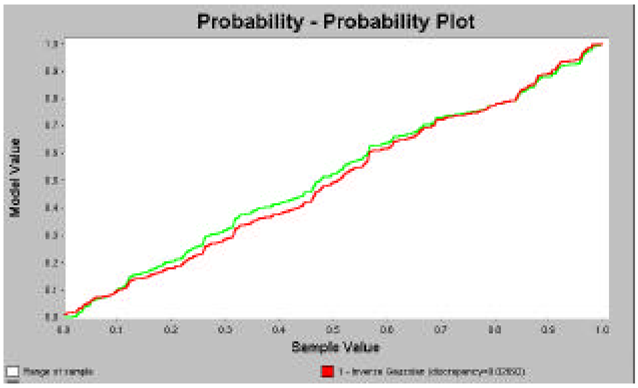



**Year 2000 Forcasts**




**Year 2001 Forecasts**

| **Distribution Parameters[1]** | | | **Descriptive Statistics** | | | | **Goodness of Fit** | |
|---|---|---|---|---|---|---|---|---|
| *?* | *ß* | *a* | *Mean* | *Variance* | *Skewness* | *Kurtosis* | *Andersen-Darling* | *Relative error[2]* |
| **808,165.9546** | **41,845.2785** | **2.34306** | **845,246.2729** | **2.82756 e 8** | **0.43286** | **2.93856** | **0.72735 (accept)** | **248.12887 = 0.03%** |

| **Distribution Parameters[1]** | | | **Descriptive Statistics** | | | | **Goodness of Fit** | |
|---|---|---|---|---|---|---|---|---|
| *?* | *ß* | *a* | *Mean* | *Variance* | *Skewness* | *Kurtosis* | *Andersen-Darling* | *Relative error[2]* |
| **748,985.163** | **35,959,897** | **2.15658** | **780,831.4266** | **2.42005 e 8** | **0.53350** | **3.07814** | **2.122172 (accept)** | **608.3219 = 0.08%** |

[1]Location (shift) parameter $g \in (-\infty, \infty)$ , scale parameter ß > 0, shape parameter a > 0 [2]Relative error is defined as the percentage difference between the sample mean and model mean

**Figs 2 and 3: Forecast distribution, P-P plot and parameters for Year 2000 and 2001 Forecasts**

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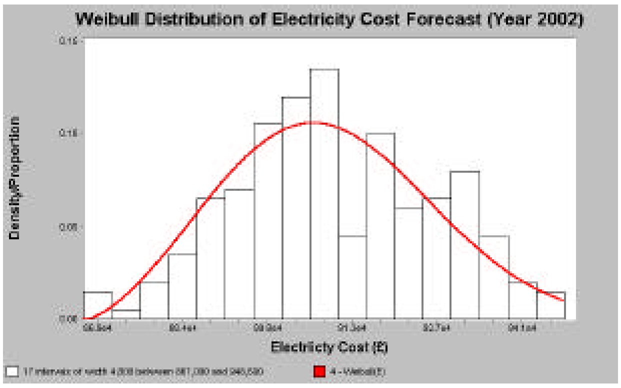
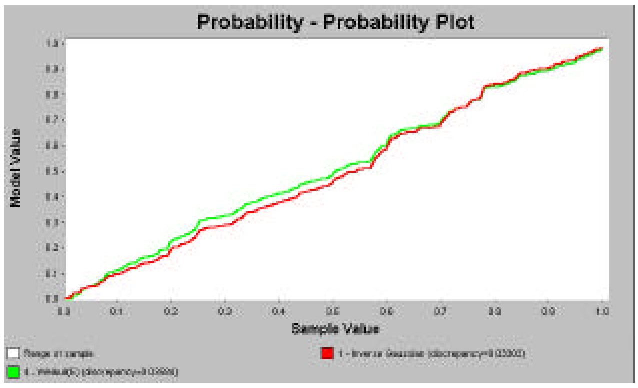
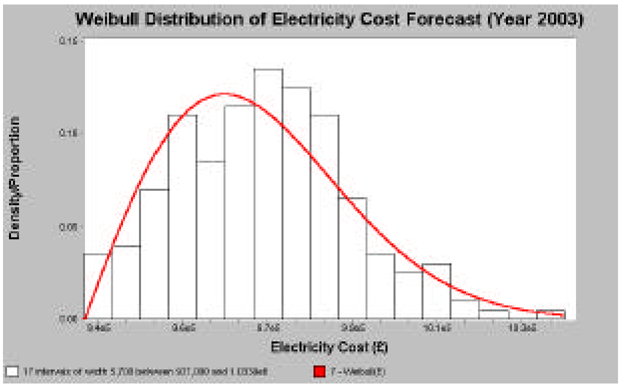
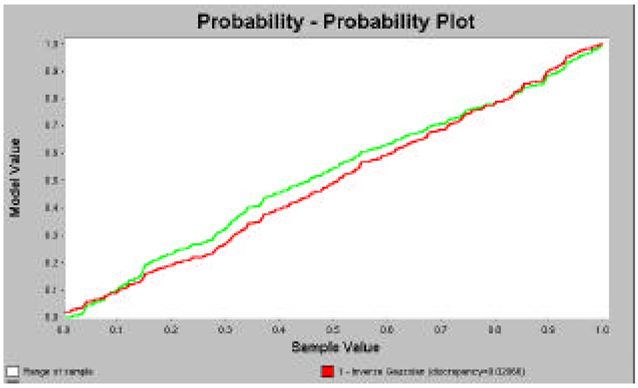



**Year 2002 Forecasts**




**Year 2003 Forecasts**

| **Distribution Parameters[1]** | | | **Descriptive Statistics** | | | | **Goodness of Fit** | |
|---|---|---|---|---|---|---|---|---|
| *?* | *ß* | *a* | *Mean* | *Variance* | *Skewness* | *Kurtosis* | *Andersen-Darling* | *Relative error[2]* |
| **867,151.545** | **47,080.0073** | **2.58241** | **908,960.18** | **3.02E+08** | **0.32285** | **2.82379** | **0.77342 (accept)** | **0.05** |

| **Distribution Parameters[1]** | | | **Descriptive Statistics** | | | | **Goodness of Fit** | |
|---|---|---|---|---|---|---|---|---|
| *?* | *ß* | *a* | *Mean* | *Variance* | *Skewness* | *Kurtosis* | *Andersen-Darling* | *Relative error[2]* |
| **937,378.097** | **40,100.2346** | **1.98164** | **972,922.3672** | **3.50989 e 8** | **0.64349** | **3.26848** | **1.15121 (accept)** | **477.35426 = 0.05%** |

[1]Location (shift) parameter $g \in (-\infty, \infty)$ , scale parameter ß > 0, shape parameter a > 0 [2]Relative error is defined as the percentage difference between the sample mean and model mean

**Figs 4 and 5: Forecast distribution, P-P plot and parameters for Year 2002 and 2003 Forecasts**

# Exploring A Building's Life Cycle Energy Through CAD

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**Summary: The ability to determine a building's life cycle energy impact is becoming increasingly important as clients, both private and government, demand greater understanding of their building's environmental impact. However, detailed life cycle energy studies are often difficult and time consuming, leading many in the industry to make crude estimates or ignore the issue altogether. Current development work with an industry partner is establishing an automated life cycle energy analysis tool. The analysis software utilises object orientated CAD data along with life cycle, embodied energy and detailed climatic data to create an individual lifetime profile for a building design. Users are able to modify their design to explore different life cycle energy scenarios through material choice, orientation and layout.**

**The tool integrates several established procedures and is one of the first applications to utilise the recently developed Industry Foundation Classes which are used for describing building components, enabling the software to run using data from any Industry Foundation Class compliant CAD package. The energy data used is from a series of detailed research projects that have been undertaken and represents one of the most up-to-date databases available.**

**This paper describes a possible approach to life cycle energy analysis through the LICHEE system which accurately analyses a standard house and produces a detailed report of building elements, materials, operational energy, embodied energy and life cycle energy to create on overall lifetime energy picture of the design.**

**Keywords. Life cycle energy, Industry Foundation Classes, Computer Aided Design**

## 1 INTRODUCTION

The built environment sector is a significant consumer of energy and of materials drawn from the environment. The conversion of raw materials to useful products is a significant contributor to Australia's greenhouse gas emissions and emission of other gas, liquid and solid waste products into the environment. Improving the environmental performance of buildings (particularly the use of energy) plays an important role in helping Australia achieve the environmental objectives set by governments, in particular the Kyoto accord on greenhouse gas emissions.

Instead of focussing on either the initial impact or the ongoing impacts of operating a building, the assessment should consider the total impacts, i.e. the whole-of-life assessment, and the full complement of impacts of buildings on the environment as developed in the concept of life cycle assessment.

Traditionally, building environmental analysis has focused on operating energy, i.e. the energy required for heating and cooling, hot water, lighting and so on. However, operating energy, while very important, is not the whole story. The energy required to manufacture, transport and assemble the building materials, i.e. the embodied energy, is also very significant, and will become more so as improvements in building design result in reductions in operating energy consumption.

Thus when evaluating the environmental impact of building materials such as concrete or timber, it is essential to do this on the basis of a whole-of-life approach, e.g. the total energy used in the building over its life, including the energy of manufacture and construction, the energy for repair, replacement and refurbishment, and the impact of material choice on operating energy. Unless this is done, sub-optimal design decisions with respect to material choices may be made. Such decisions do not necessarily lead to a minimisation of whole-of-life impacts such as greenhouse gas emissions.

The benefits to the user (e.g. designers) of a computer system for life cycle energy assessment are in being able to compare the environmental impact (as measured by the total energy required to obtain the raw materials, manufacture the building products, transport them, construct the building and their impact on the operating energy) of one design to another to assist in determining the most environmentally friendly overall design. Such an approach could include any emissions etc generated by the construction and operation of a building.

In order to estimate the amount of energy embodied in a building and the resultant greenhouse gas emissions generated through energy consumed in these processes, prototype software to test the feasibility of such an approach was developed by CSIRO for the CAD package APDesign to calculate the material quantities, embodied energy and greenhouse gas emissions of the initial construction directly from a 3D CAD model. This model provided quantitative values to assist in determining the environmental impact (including $CO_2$ emissions resulting from energy used to create materials used in a building) of alternative designs and building materials at the design stage of a building.

The principles developed through this prototype software were utilised to create a procedure based on ArchiCAD which design professionals can use to assess alternative designs in terms of environmental impact as measured by the amount of energy required to construct and operate a building over its lifetime with the provision for additional impacts to be readily added as usefulness and data availability requires it. The procedure is called LICHEE (**LI**fe **C**ycle **H**ouse **E**nergy **E**valuation).

This paper describes a possible approach to life cycle energy analysis through the LICHEE system which accurately analyses a standard house and produces a detailed report of building elements, materials, operational energy, embodied energy and life cycle energy to create on overall lifetime energy picture of the design.

## 2 LIFE CYCLE ASSESSMENT

Life cycle assessment as defined by ISO 14040, attempts to assess the impact on the environment of any product (including buildings) from "cradle to grave", i.e. from obtaining the raw materials from which to create the product to its disposal at the end of its life (ISO 1997). The most significant impact is usually in the use of energy and its resultant greenhouse gas (mostly $CO_2$), with the life cycle aspect including the energy to create the product and operate it throughout its life. A full life cycle assessment includes all emissions rather than just energy and greenhouse gas emissions, including the impacts of pollutants released to the air, water and land during creation and operation.

To be able to quantify environmental impacts resulting from the construction of a building, the quantities of materials must first be estimated through a process of disaggregation to a level of detail which allows for the separation of components into their principal materials. Impact intensities of each material can then be multiplied by the quantities of individual materials and the products aggregated to obtain the total for each material, element or whole building. Consistent and reliable databases of intensities are an essential part of any life cycle assessment and it is usually where the majority of work is concentrated. A database of energy intensities has recently been derived from input-output tables and other national and international studies and is used within this study to demonstrate the principle of life cycle assessment.

Lifetimes of the materials chosen can significantly affect the total environmental impacts through materials and components having to be replaced at intervals throughout the life of a building. Thus the life cycle approach requires estimation of durability capabilities of building materials which affect the repair and replacement regimes of the components of a building. Some materials failures require replacement of other materials which have not reached the end of their useful life. Life cycle models must include provision for such replacements to give the correct overall environmental impacts of a building over its whole life.

The life cycle operating energy requires calculation of the annual energy consumption (or environmental impact) using a reliable and proven energy evaluation technique. The operating energy calculations require knowledge of material properties such as heat transfer rates to estimate heating and cooling requirements of a building. The calculated energy of construction, operating energy and life cycle replacements and maintenance are then combined to estimate the life cycle energy. Results are usually presented as performance indicators to readily analyse and assess the impacts.

A life cycle assessment of building energy is thus a comprehensive approach which demands knowledge of construction such as quantities of materials combined with many properties of materials such as embodied energy, durability, and heat transfer characteristics. Any practical approach to life cycle energy estimation requires a fully integrated system which can be readily invoked by a user to compare alternatives. The first requirement is to obtain the relevant information from the drawings / plans for a building.

## 3 INDUSTRY FOUNDATION CLASSES (IFC)

One of the great disadvantages of many life cycle assessment procedures for buildings, or indeed any system, is the need to quantify and enter data about a building into the assessment process. This can be very time consuming and as a design progresses the updating of data and tracking of changes can become onerous and error prone.

Automation of data entry and utilising existing sources of information are of key importance if life cycle assessment is to be generally adopted. The next generation of CAD systems offer a possible avenue for this data transfer. Traditionally, CAD drawings have been simple line representations of a building with no associated information as to what the lines actually represent, that is, walls, windows, roofs, etc. However, object orientated CAD systems do contain such information and provide the opportunity to develop automated analysis software.

The Industry Foundation Classes (IFCs) currently being developed and implemented world-wide for information exchange from proprietary CAD systems is the future of data transfer platforms (Wix & Liebich 1997). The IFCs are a set of electronic specifications that represent objects that occur in constructed facilities (including real things such as doors, walls, fans, etc. and abstract concepts such as space, organization, process etc.). These specifications represent a data structure supporting an electronic project model useful in sharing data across applications and were adopted in the LICHEE system.

Each specification is called a 'class'. The word 'class' is used to describe a range of things that have common characteristics. For instance, every door has the characteristics of opening to allow entry to a space; every window has the characteristic of transparency so that it can be seen through. Door and window are names of classes and these classes are termed Industry Foundation Classes or IFCs. The major advantage of utilising IFC technology is that it allows analysis of drawings produced from any IFC compliant system. Many of the major CAD vendors are moving towards IFC compliance. Identification of every object in a CAD drawing by class allows analytical software calculating building performance measures such as embodied energy and operational energy to obtain almost all of the desired characteristics directly from the CAD drawing.

## 4 LIFE EXPECTANCY AND DURABILITY

The life expectancy of building components is a key aspect of environmental indicators as no building remains maintenance and refurbishment free after construction. Estimating the actual life expectancy of a building component and associated materials is not necessarily the same as the component and materials possible life expectancy which is determined through their durability. Other life expectancy factors often interact with a component to shorten their life expectancy, such as, material interaction, environmental impacts and operational conditions.

The effective durability of materials is usually controlled by the building components they are associated with and/or the building itself. For example, aluminium windows are made up of extruded aluminium for the frame, glass, weather-stripping and gaskets and window hardware (locks and latches), as shown in Figure 1. Each material has their own individual life expectancy, but some are reduced by the life expectancy of another critical material, while the component itself may have its overall life expectancy reduced by the life expectancy of the building it is installed in.

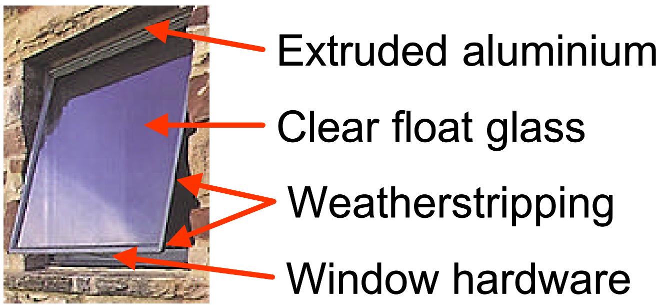


**Figure 1Material components of an aluminium window**

Table 1shows the four main materials in an aluminium window and their relative life expectancies. The life expectancies listed are considered to be ‘real world’ values rather than theoretical values which tend to be higher, and were obtained from the *HAPM Component life manual* (Housing Association Property Mutual, 1994). Good practice maintenance regimes are also employed which accounts for the relative short life times of the weather stripping and hardware. The effective life expectancy of the window is determined by the life expectancy of the aluminium frame which is considered to be a critical material, that is, once it fails and requires replacement all other materials will also be replaced regardless of their condition. Thus, if this window is installed into a building with an 80 year life expectancy it can be assumed that the window will be replaced twice, once at the 30 year mark and again at 60 years.

**Table 1. Durability effectiveness of an aluminium window**

| Component – Aluminium Window | | | |
|---|---|---|---|
| **Materials** | **Life expectancy (years)*** | **Replacements over 80 years** | **Durability effectiveness** |
| Extruded aluminium | 30 | 2 | 89% |
| Clear float glass | 100 | 2 | 27% |
| Weather-stripping/gaskets | 10 | 7 | 100% |
| Window hardware | 10 | 7 | 100% |

* Source: Component life manual (HAPM, 1994)

During the life time of the window maintenance procedures are carried out which see the weather-stripping and window hardware replaced every ten years or seven times over the life of the building. The glass in the windows has a long life expectancy, but because it is replaced when the frame is replaced, its life expectancy is reduced to that of the frame and is also replaced twice over the building's life. This reduction in life expectancy can be expressed as a percentage of durability effectiveness.

From Table 1 it can be seen that the aluminium frame has a durability effectiveness of 89% as it is affected by the 80 year life expectancy of the building which reduces the second replacement window's life to 20 years. The glass has a durability effectiveness of 27% as it is reduced by the two frame replacements and the building's life expectancy. The weather-stripping and hardware both rate 100% as their full life expectancy is achieved on all replacements.

This approach gives an indication of the effect that durability of one material within a component can have on the component's other materials. Creating components whose constituent materials have life expectancies as close as possible is an effective solution. This reduces the need for replacement maintenance and helps maximise the durability effectiveness of all materials.

The life expectancy of 80 years for the building was selected as representing a 'reasonable' life expectancy for a modern residential building within Australia. Determining the life expectancy of a building is not straightforward and is often subject to a range of factors. Statistical data is virtually non-existent given Australia's relatively young urban environment. ISO 15686 outlines three main types of obsolescence: functional, technological and economic (ISO 2000). Any of these could be attributed to the determination of the life expectancy for a specific residential building, but inevitably it will effect the life expectancy of certain components within the building which are not obsolete, as with the example window.

The LICHEE system does factor in estimates of material and component life times along with maintenance regimes that usually exist. However, these factors are fairly broad and do not take account of local environmental conditions that may alter the life expectancy of building components. The durability of materials within certain environments is an area of research that has been undertaken by CSIRO and a database is being developed. It is envisaged that this data will be incorporated into the software to provide important additional durability information to designers.

## 5 LICHEE

The LICHEE system is not a single stand-alone package but a series of intercommunicating programs. Some of these programs, such as the Nationwide House Energy Rating Software (NatHERS), have been developed over many years and it is the integrating of these sophisticated individual programs into an all-encompassing energy analysis tool that is the real power of the LICHEE system. The main components of the system and the data flow between them (as shown in Figure 2) are:

- ArchiCAD - an object-oriented CAD architectural system with an IFC export capability. This provides a simple method to export data in a vendor neutral way.
- The IFC processor - the core of the LICHEE system which takes the information exported from ArchiCAD, extracts the necessary information from the IFC file for both the NatHERS and embodied energy calculations, then exports the data, controls the calculation of both embodied and operating energy and initiates display of the results.
- Material quantities calculator - converts the component dimensional information produced by the IFC converter into quantities of materials. This information includes details of the component parts of the house for which they are used.
- Life cycle energy calculator - uses the material quantity information produced by the previous component to calculate the lifetime embodied energy, $CO_2$ and mass amounts. The embodied energy software uses material quantities, along with data on embodied energy coefficients and life times of the various house components. It also uses formula sets that allow conversion of quantities of, for example windows, into quantities of glass, aluminium, etc. All this data is combined to calculate the life time energy required by the house.
- NatHERS - Operational energy is calculated using the standard version of the simulation engine along with the building's star rating. It should be noted that the only components of operational energy calculated by this engine are heating and cooling energy requirements, i.e. the amount of energy required to be delivered to or extracted from the space to maintain the thermostat settings. The energy actually consumed by the heating and cooling equipment is estimated from the requirement by dividing by an appropriate efficiency or coefficient of performance. Other energy consumers such as hot water, lights, etc, are not considered by the NatHERS engine.
- Reporting component - reads the file produced by the processing programs and produces various tables and graphs, and summary information. It also generates a short printed report.

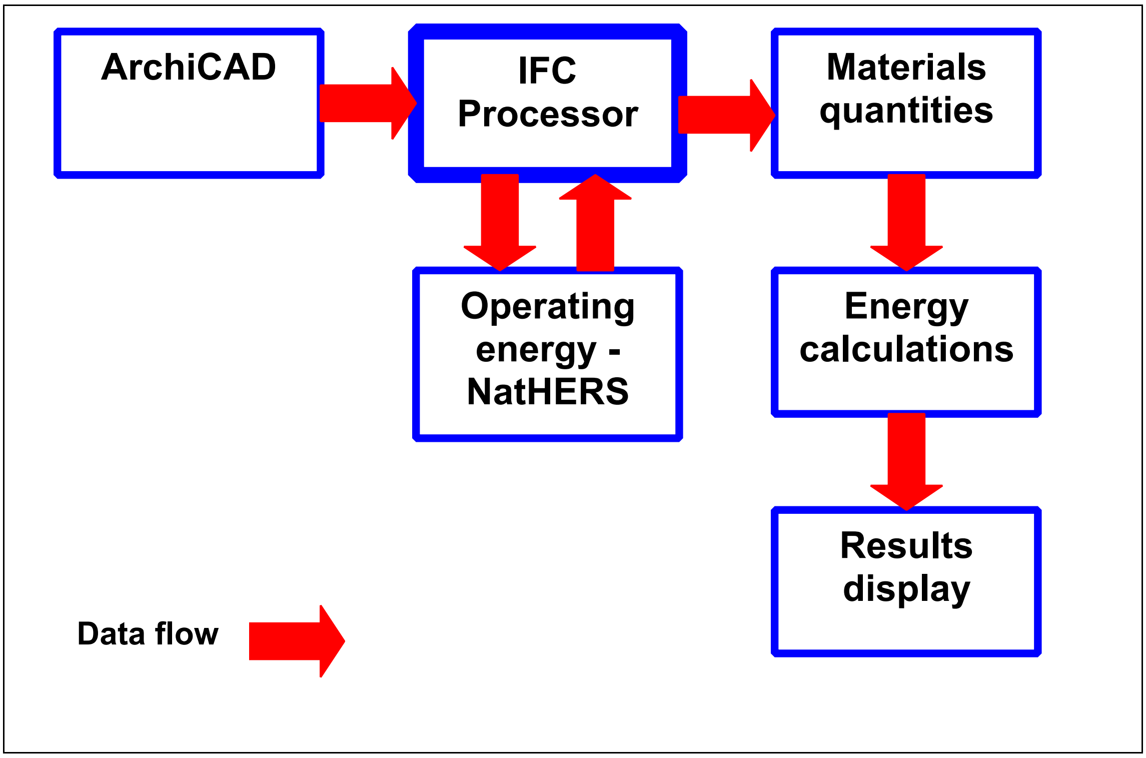


**Figure 2Outline of software modules**

## 6 OUPUT

The reporting component of LICHEE provides the following outputs:

- Summary of results,
- Progressive graphs of life cycle energy for material groups, element and materials, in total or individually over the building lifetime,
- Cumulative graph of life cycle energy over the building lifetime,
- Progressive and cumulative tables of life cycle energy for material groups, element and materials, in total or individually over the building lifetime,
- Graphs of the total life cycle embodied energy, $CO_2$ and mass by breakdown of material groups, element and materials, and
- Tables of the total life cycle embodied energy, $CO_2$ and mass by breakdown of material groups, element and materials.

Many of the graphs and tables have the facility to click on any material group, element or material to display either the element from which it came or the materials which make up that element. Figure 3 and Figure 4 show examples of the type of graphs that the system is able to produce. Both graphs were produced from a design for a typical brick veneer home.

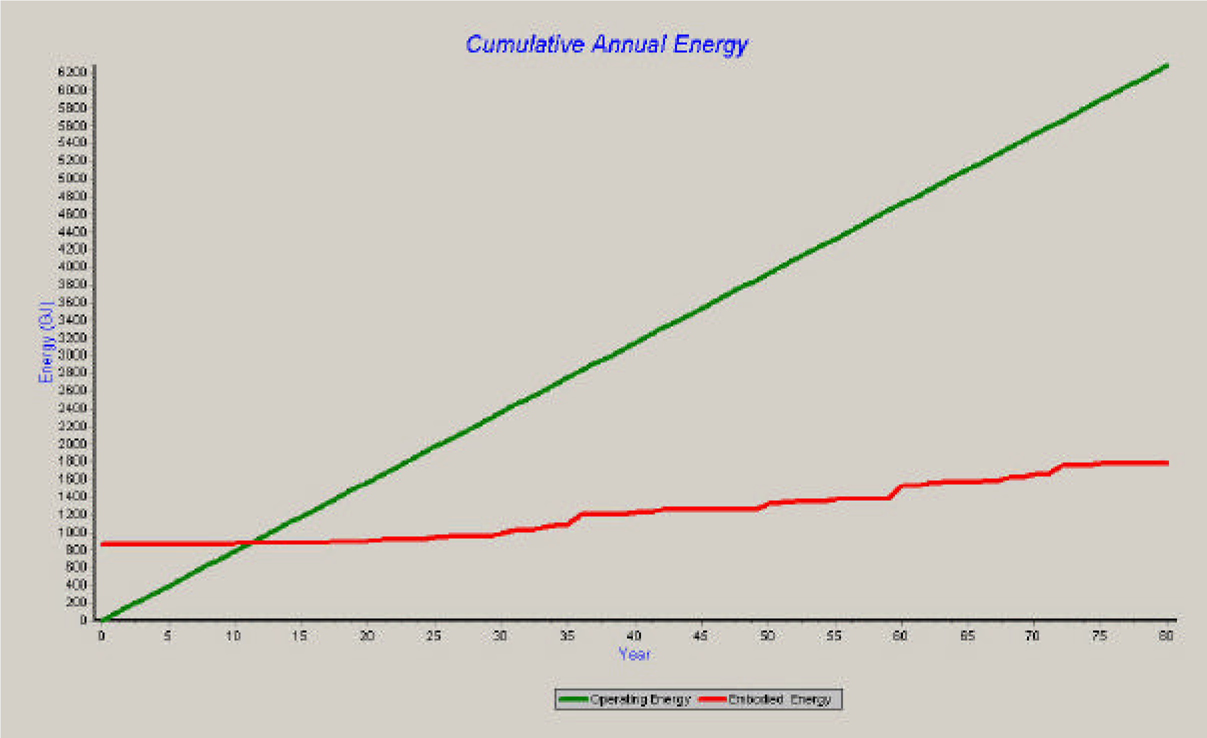


**Figure 3 Cumulative annual energy graph**

Figure 3 shows the cumulative annual energy of the house over its designated 80-year lifetime. It is interesting to note that the operating energy overtakes the embodied energy of the house after approximately 11 years and over the lifetime of the house the operating energy is more than triple the entire embodied energy. Nevertheless, the embodied energy is still a significant contributor to the dwelling's total energy consumption and demonstrates the importance of total lifecycle analysis. The steps in the embodied energy line represent the maintenance and repair cycles of a typical building. The significant jumps at around the 35 and 60 year mark represent major repair cycles when many significant building components need replacement at the end of their effective life time.

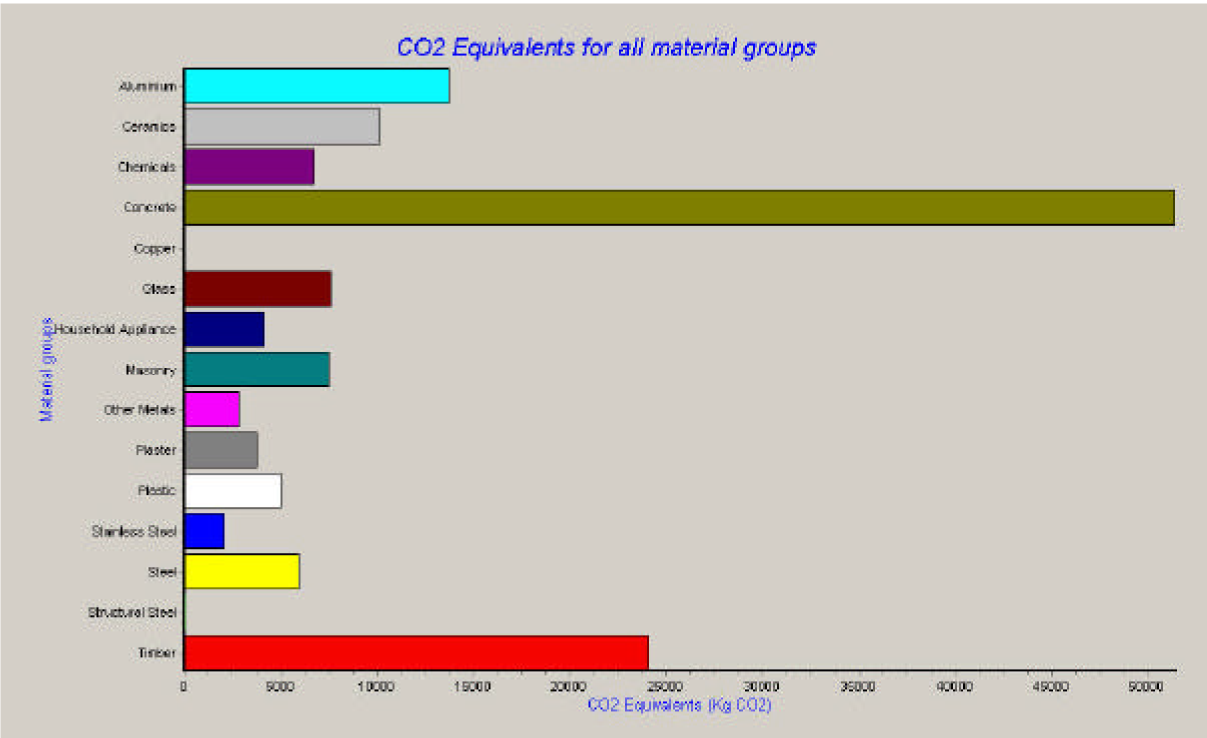


**Figure 4 $CO_2$ emissions for material groups graph**

Figure 4 shows a detailed graph identifying the contribution of the material groups to the life cycle $CO_2$ emissions. Similar results can be displayed for embodied energy or mass by individual materials and building elements.

## 7 FUTURE DIRECTIONS

The LICHEE system is presently restricted to energy calculations for residential buildings. However, it demonstrates the ability to perform analysis calculations from CAD data without the need to re-enter much of the information. Building upon such a system is relatively easy provided the data is available. For example, it is envisaged that the system will be expanded to incorporate a range of environmental considerations and enable a full environmental life cycle assessment to be performed. These could include acidification and nutrification potentials, other greenhouse gases in addition to $CO_2$, ozone depletion, smog, human toxicity such as carcinogens and solid waste.

Improvements in its handling of operating energy aspects are also seen as an important area. These would include:

- Selecting the type of heating and cooling system being used, including the option of having none. This would impact on the relative efficiencies for the various systems which is important in determining the energy consumed.
- Selecting the type of hot water system used, which again is important for efficiency.
- Setting the number of occupants in the house and selecting a user profile which determines when the house is occupied. For example, elderly people may be home all day and have a higher thermostat setting, whereas a working couple will be away for much of the day during the weekdays.
- Operating pollutant emissions related to choice of energy source, mainly electricity or gas.

## 8 CONCLUSIONS

The importance of life cycle assessments is increasing within a broad range of industry areas. The ability to quickly and accurately perform such assessments is going to be essential in the years to come. However, life cycle assessments are only as good as the databases that are behind them. It is imperative that accurate and reliable databases be developed for a large range of environmental factors. Material performance including durability are an intrinsic part of these databases and have a significant impact on life cycle assessment within the building industry.

The LICHEE system has been developed to allow designers quick and detailed life cycle analysis of their designs by utilising an existing CAD system coupled with a vendor free data representation format, all with little additional input. This application also demonstrates the possibilities for other building analysis work to utilise the IFC technology to gather building information and perform calculations.

The usefulness and importance of integrating analytical software packages with databases of material properties including life times based on durability has been demonstrated for a specific performance indicator, life cycle energy of houses. It is hoped that expansion of the current system will see a greater variety of buildings being covered and a greater breadth of environmental impacts being analysed. This would then provide a comprehensive environmental analysis tool that would greatly simplify the assessment of environmental impacts of the built environment.

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## 10 ACKNOWLEDGMENT

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# Integrating LCA And LCC To Select Cost-Effective Green Building Materials - The BEES Approach

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**Summary: The BEES (Building for Environmental and Economic Sustainability) software implements a new technique for balancing the environmental and economic performance of building products. The tool is based on consensus standards and designed to be practical, flexible, and transparent. Version 2.0 of the decision support software, aimed at designers, builders, and product manufacturers, includes environmental and economic performance data for 65 building products. The purpose is to support purchasing decisions by providing life-cycle-based information often lacking in 'green' product selection. The intended result is a cost-effective reduction in building-related contributions to environmental problems.**

**Keywords: Building product performance; Life Cycle Assessment (LCA); life-cycle costing; environmental performance; multiattribute decision analysis**

## 1 INTRODUCTION

How do *you* identify environmentally preferable products? Designers, builders, and researchers are increasingly asked to address the issue of 'green' building materials. Is a product environmentally preferable if it has recycled content? Is it not preferable if it offgasses during use? Are mainstream products always less preferable than products marketed and perceived as 'environmentally friendly'? Do environmentally preferable products always cost more? Not necessarily.

The latest version of the BEES (**B**uilding for **E**nvironmental and **E**conomic **S**ustainability) software implements a new technique for selecting cost-effective, 'green' building products (Lippiatt 2000). The tool is based on consensus standards and designed to be practical, flexible, and transparent. Version 2.0 of the decision-support software – aimed at designers, builders, and product manufacturers – includes environmental and economic performance data for 65 generic building products.

BEES is developed in the United States by the NIST (National Institute of Standards and Technology) Building and Fire Research Laboratory with support from the U.S. EPA Environmentally Preferable Purchasing (EPP) Program and the White House-sponsored Partnership for Advancing Technology in Housing (PATH). The EPP Program is charged with carrying out Executive Order 13101, 'Greening the Government Through Waste Prevention, Recycling, and Federal Acquisition', which encourages Executive agencies to reduce the environmental burdens associated with the $200 billion in products and services they buy each year, including building products. BEES is being further developed as a tool to assist the Federal procurement community in carrying out Executive Order 13101.

## 2 METHODOLOGY

BEES measures the environmental performance of building products using the internationally-standardized and science-based life-cycle assessment approach (ISO 1997, 1998, 2000). All stages in the life of a product are analyzed: raw material acquisition, manufacture, transportation, installation, use, and recycling and waste management. Up to ten environmental impacts are measured across these life-cycle stages: global warming, acid rain, resource depletion, indoor air quality, solid waste, eutrophication (the unwanted addition of mineral nutrients to the soil and water), ecological toxicity, human toxicity, ozone depletion, and smog. Due to its comprehensive, multi-dimensional scope, life-cycle assessment accounts for shifts of environmental problems from one life-cycle stage to another, or one environmental medium (land, air, or water) to another. The approach highlights the tradeoffs that must be made to genuinely reduce overall environmental impacts.

BEES measures economic performance using similar life-cycle thinking. Economic performance is measured using the ASTM standard life-cycle cost method, which covers the costs of initial investment, replacement, operation, maintenance and repair, and disposal (ASTM 1994). The life-cycle cost method sums these costs over a fixed period of time, known as the study period. Alternative products for the same function, say floor covering, can then be compared on the basis of their life-cycle costs to determine which is the least-cost means of covering the floor over the study period.

To combine environmental and economic performance into an overall performance measure, BEES uses the ASTM standard for Multiattribute Decision Analysis (ASTM 1995). The BEES user specifies the relative importance weights used to combine environmental and economic performance scores and may test the sensitivity of the overall scores to different sets of relative importance weights. Supporting data and computations are documented.

## 3 CASE EXAMPLE

So how can *you* use BEES to compare the environmental and economic performance of competing products? Let's run through an example. Suppose we're considering two floor coverings: (1) a broadloom nylon carpet installed using a conventional glue, a mainstream alternative, and (2) a broadloom carpet made from PET (recycled soft drink bottles) and installed using a low-VOC glue (a glue emitting relatively low levels of volatile organic compounds), a product promoted as an environmentally friendly alternative.

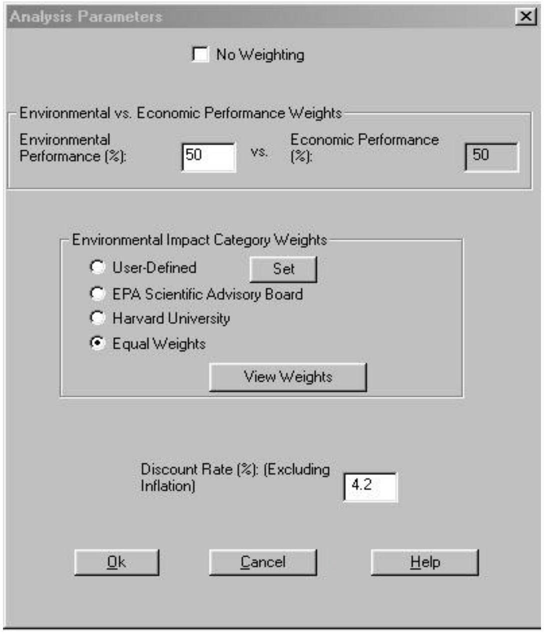


**Figure 1. Setting BEES analysis parameters.**

The first step is to set our analysis parameters using the BEES window shown in Fig. 1. If we do not wish to combine environmental and economic performance measures into a single score, we can select the 'No Weighting' option and still compute disaggregated BEES results. Otherwise, we need to set importance weights. In this example, environmental performance and economic performance are of equal importance so both are set to 50%. Next, we need to set relative importance weights for the environmental impact categories included in the BEES environmental performance score. We select the 'Equal Weights' set, assigning equal importance to all impacts. Our last parameter is the real discount rate used to convert future building product costs to their equivalent present value. Here, we accept the default rate of 4.2%, the rate mandated by the U.S. Office of Management and Budget for most Federal projects (OMB 1992, 2000).

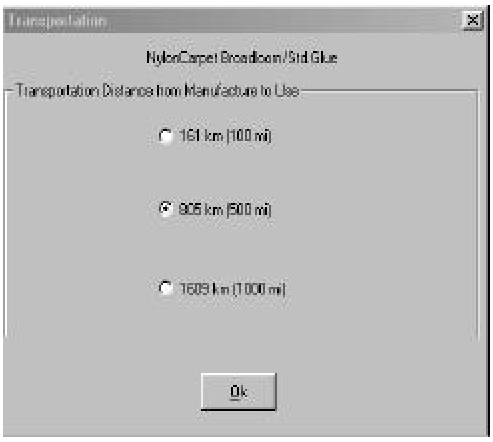


**Figure 2. Setting transportation parameters.**

Next, we need to set one last parameter for each of our floor covering alternatives – the transportation distance from the manufacturing facility to the building in which the product will be installed. This parameter lets BEES compute an environmental performance score accounting for the significance of using locally-produced products. As illustrated in Fig. 2, we have selected a transportation distance of 805 km (500 mi) for our nylon carpet alternative.

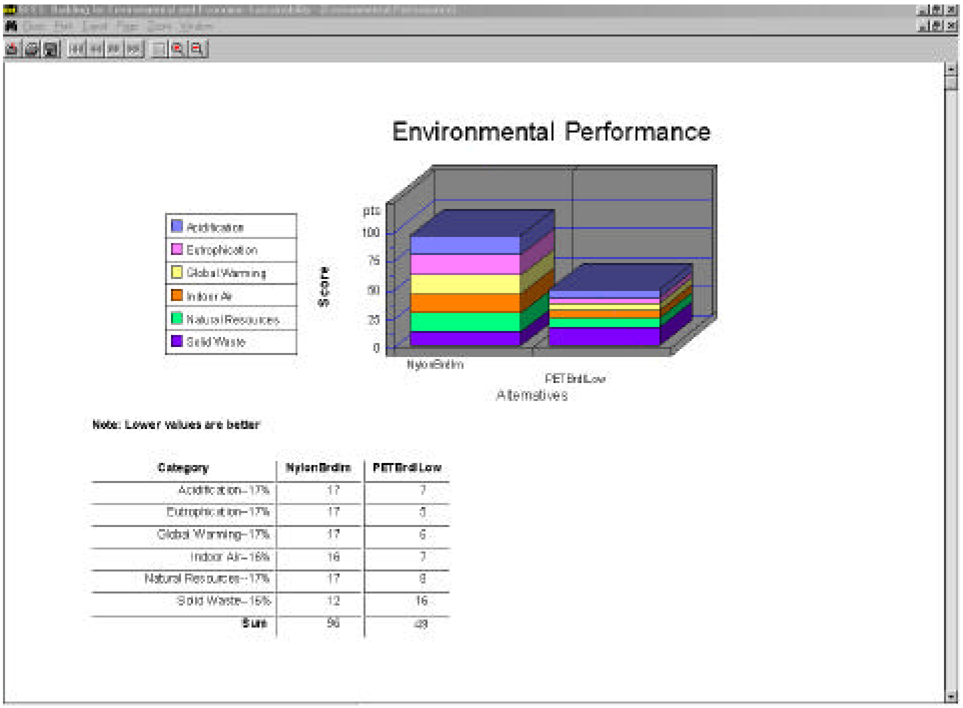


**Figure 3. Viewing BEES environmental performance results.**

Now we are ready to compute and view BEES results. Figure 3 shows the BEES Environmental Performance Results displaying the weighted environmental performance scores for our example in both graphical and tabular form. Lower values are better; if a product performs worse with respect to all environmental impacts, it receives the worst possible score of 100 points. In our example, the nylon broadloom carpet receives a total score of 96 points and the PET broadloom carpet a total score of 49 points. The figure breaks down the weighted environmental score by its six contributing, weighted scores for acidification, eutrophication, global warming, indoor air, natural resource depletion, and solid waste. As shown, PET carpet performs better on all impact categories except solid waste. Displayed on the table, next to each impact category, is its assigned relative importance weight.

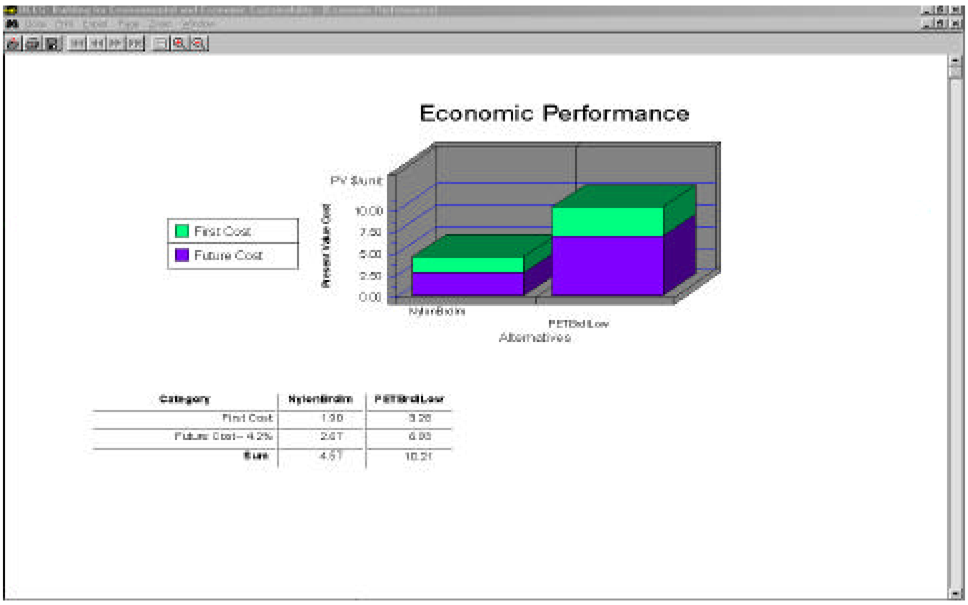


**Figure 4. Viewing BEES economic performance results.**

Figure 4 shows the BEES Economic Performance Results for our example, which gives first costs, discounted future costs, and their sum, the life-cycle cost. The figure shows that PET broadloom carpet has a higher life-cycle cost ($10.21 in present value dollars per 0.09 $m^2$ of installed carpet, or $10.21 per $ft^2$, compared with $4.57 for nylon), with both a higher first cost and higher future costs (due to a higher and more frequently-occurring replacement cost). Thus, based on our assigned discount rate of 4.2% (displayed in the table next to the future cost category), PET broadloom carpet scores better environmentally, while nylon broadloom carpet scores better economically.

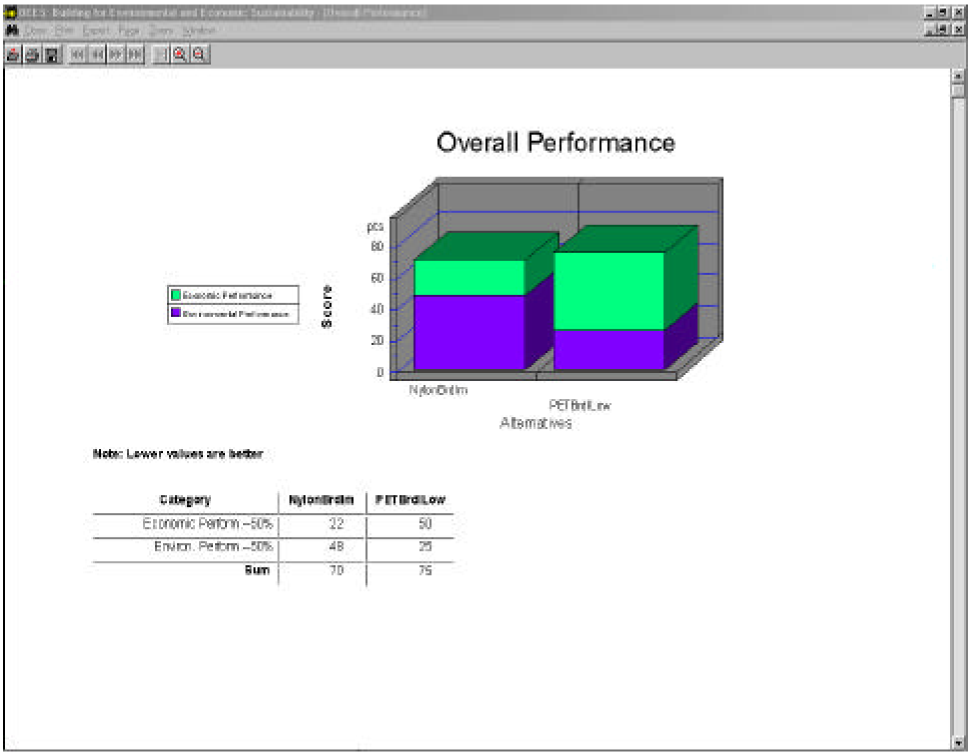


**Figure 5. Viewing BEES overall performance results.**

The overall performance score gives us a way to combine and balance the environmental and economic performance scores. Figure 5 shows the BEES Overall Performance Results based on our equal weighting of environmental and economic performance. It displays the overall performance score for each product alternative, which is the sum of its weighted environmental and economic performance scores. Displayed in the table, next to each performance category, is its assigned relative importance weight. We can see from this figure that nylon broadloom carpet receives a score of 70 points and PET broadloom carpet a score of 75 points. Thus, based on our analysis parameters, nylon broadloom carpet installed with conventional glue is slightly preferable overall to PET broadloom carpet installed with low-VOC glue. Note that besides the summary graphs shown here, BEES also offers detailed graphs for each environmental impact (e.g., reporting grams of carbon dioxide each product contributes to the global warming impact), which help pinpoint the 'weak links' in a product's environmental life cycle.

## 4 CONCLUSION AND FUTURE OUTLOOK

Applying the BEES approach to the scores of other products included in BEES 2.0 (including framing, exterior and interior wall finishes, wall and roof sheathing, ceiling and wall insulation, roof and floor coverings, slabs, basement walls, beams, columns, parking lot paving and driveways) leads to several general conclusions. First, environmental claims based on single impacts, such as reduced global warming alone, should be viewed with skepticism. These claims do not account for the fact that one impact may have been improved at the expense of others. Second, assessments must always be quantified on a *functional unit* basis as they are in BEES, so that the products being compared are true substitutes for one another. One roof covering product may be environmentally superior to another on a kilogram-for-kilogram basis, but if that product requires twice the mass as the other to cover one square meter of roof, the results may reverse. Third, a product may contain a high-impact constituent, but if that constituent is a small portion of an otherwise relatively benign product, its significance decreases dramatically. Finally, a short-lived, low first-cost product is often not the cost-effective alternative. A higher first cost may be justified many times over for a durable, maintenance-free product. In sum, the answers lie in the tradeoffs.

BEES will be expanded and refined over the next several years. First, many more products will be added to the system so that entire building components and systems can be compared. To that end, manufacturers are encouraged to submit brand-specific performance data through the new *BEES Please* program (contact: blippiatt@nist.gov). Second, more environmental impacts, such as habitat alteration, are under development for incorporation into future versions of BEES. Finally, U.S. region

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specificity and greater flexibility in product specifications (e.g., useful lives) are being incorporated. The intended result is a cost-effective reduction in building-related contributions to environmental problems.

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# The Application Of Risk Methods For The Economic Viability Of Air Conditioning Systems

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**Summary: The installation and subsequent operation of any mechanical air conditioning systems represents a significant capital expenditure. It is therefore important to assess the likely future costs of such systems at the outset so that an informed decision can be made regarding the selection of an appropriate system. Whole Life Cycle Costing techniques can be used to forecast these costs, but the integration of stochastic methods will provide the analyst with the ability to quantify the uncertainty associated with these forecasts. This paper reports on the development of a stochastic model, which analyses the economic viability of four different air-conditioning systems. A stochastic modelling software package was utilised to analyse complex data sets to generate the parameters of the theoretical distribution functions that best represent the cost of installing, operating and maintaining four different air-conditioning systems over a 20 year period. The Anderson-Darling goodness of fit validated the generated distributions. The resulting probability distributions enable the analyst to define costs that can be associated with prescribed levels of confidence, thus allowing a quantification of the exposure to budget overruns and financial risk in acquiring new air conditioning systems. This is essential for preparing for annual management plans and budgets.**

**Keywords: Risk, probability, air conditioning systems, economic viability**

## 1 INTRODUCTION

Mechanical services are one of the fasted developing forms of technology within building with increasing investment of over 15% in the last 10-15 years (Cassidy 2000). The cost for mechanical services can fall between 10 and 40 per cent of the total construction cost, depending upon the desired function, sophistication, complexity and prestige of the building. It is not just the initial capital cost that a designer has to take into account when choosing the mechanical services systems, the operational and maintenance costs and life expectancy have to be considered as well. Mechanical services are usually the most expensive element of any building as it involves not only major capital investment, but also significant expenditure on maintenance, renewal and operation. The relationship between building function and M&E costs was identified by Brown (1987). The study also found the allocation of the total services cost among the various services elements could not be predicted accurately from known the buildings function. Therefore, only totals for the complete M&E services could be predicted as a percentage of the total building cost. Building form is another issue that has been investigated to see if it has a significant bearing on the cost of M&E services. Brandon (1978) identified a number of descriptors of building form that may be useful for determining M&E services cost. These include; plan shape, number of storeys, boundary coefficient, average storey height, percentage of glazed area, and plan compactness. However, Swaffield and Pasqure (1999) found that other forms of descriptors could be more useful for M&E services cost estimates. Horizontal distribution volume and internal cube were found to be the most useful variable.

Operational and maintenance costs play an important role in the procurement of mechanical services. Environmental issues need also to be investigated, such as equipment compliance with current and future legislation in terms of $CO_2$ emissions, Ozone depletion and health and safety. These will have a bearing on the initial capital cost and the whole life cycle costs of the various systems. Most of the existing methods for forecasting these costs are based on deterministic methods ignoring the inherent uncertainty and variability of the real world, for which a probabilistic methodology is better suited. For this purpose, simulation is widely accepted and recognised by practitioners and researchers as a modelling tool for dealing with uncertainty. It is also widely acknowledged that the quality of a simulation model's results is strongly correlated to the quality of the input probability distribution functions. There is a need to assess the viability of installing AC systems. This is because decisions made during the design can have a significant impact on the future operating and maintenance costs of buildings. Hence, in this paper a first attempt is made to present a new concept for modelling and analysing the economic viability of air-conditioning systems using applied probabilistic methods. It is based on assessing the characteristics of the probability distribution of the unit cost per square meter for acquiring and operating four different AC systems.

## 2 MODELLING THE WHOLE LIFE COST OF AIR CONDITIONING SYSTEMS

From purely financial perspective total costs for acquiring AC systems can be defined under four different cost centres. These are; initial capital costs; major asset replacement costs; sub-asset replacement costs and planned preventative maintenance costs and energy costs (Building 2000). The model developed in this work is based on the assumption that these cost centres and the life expectancy of the AC components are randomly distributed according to one of the theoretical distribution forms. This requires then that each cost centre element be treated stochastically and the cost of operating AC systems should be represented as probability distribution function (PDF). The total costs of operating AC systems are considered to be variable from one year to another, depending on inflation and interest rates. Hence if the PDF or cash flow profile of each AC system acquisition and running is known or can be simulated, the total NPV of acquiring and operating AC systems can be estimated using the following equations:

$$NPV = P_c + \sum_{i=1}^{n} \frac{(AR_c + SR_c + EG_c + PM_c)}{(1+r)^i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

Where $P_c$ = the initial acquisition costs at time $i = 0$, $AR_c$ = major asset replacement costs, $SR_c$ = sub-asset replacement costs. $PM_c$ = planned preventative maintenance costs and $EG_c$ = energy costs relating to the running of AC systems.

The initial capital costs of AC systems include the costs for equipment, installation, commissioning, design fees and legal fees. The energy costs include all power consumed in relation to operating the equipment after installation. Energy efficiency of AC systems is becoming increasingly important as it can account for the largest single element of the energy requirement of a building. Also, in the near future, energy tax could possibly be introduced. It may be more cost effective to install a system with higher initial cost but which provides greater energy efficiency. Operational and maintenance costs including cleaning, servicing and repair. Maintenance cost is often over-looked in these kind of calculations and can be very costly dependent on the complexity of the AC system. Flexibility of the installed AC systems should also be assessed in relation to the occupier or user, the operator and/or maintenance personnel. Many systems have standard controls, which are simply in concept yet sophisticated in nature and can combine user friendliness with full technical diagnostics. Replacement cost (or residual costs) includes any planned replacement of plant or other components at the end of useful life of the system. In this research, replacement costs are subdivided into major asset replacement (items of plant and equipment that have life span over 10 years) and sub-asset replacement (this includes the periodic replacement of AC components, like motor drivers etc.).

The second term of equation 1 can also be expressed in the following formula:

$$Ac_{vr}|_q = \sum_{i=1}^{n} \frac{(AR_c|_q + SR_c|_q + EG_c|_q + PM_c|_q) \times P\{q\}}{(1+r)^i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

where q = 1…….,m represents possible events (costs) , $AR_{c,}$ $SR_c$ $EG_c$ and $PM_c$ are running costs respectively in period I due to the occurrence of q, P(q) is yearly probability of occurrence of a particular event (in this case the probability that q occurs) ), $AC_{vr}$ is the expected cost in period i and i is time or period of analysis

Where $AC_{vr}$ is variable cost of running the acquired AC system. Equation 2 expresses the yearly stochastic nature of the AC acquisition and operation. This equation is useful for carrying out a sensitivity analysis to find out the parameters that influence the viability of AC systems. As described above $AR_c$, $SR_c$, $EG_c$; $PM_c$ and r are random quantities, and hence, $AC_{vr}$ is a function of several random parameters. The composition of $AR_c$, $SR_c$, $EG_c$ and $PM_c$ may have a significant influence on the form of the PDF of yearly and total NPV computation.

## 3 DATA AND RESEARCH METHODOLOGY

To study the economic viability of AC systems, NPV financial indicators are suggested as an effective measure for aiding in the selection an AC system. To compute NPV values, accurate information regarding the input determinants in equation 1 and 2 is required. The state of knowledge of AC systems over their expectancy life does not provide sufficient data or knowledge to assist financial analysts in estimating deterministically the total cost of acquiring and running AC systems. Because of this lack of information Monte Carlo simulation is used in this paper to evaluate the economic viability of AC systems. Monte Carlo simulation facilitates the use of probability distributions for each of the variables in equation 1 and 2, enabling the simulation model to account for uncertainty and variability in the cost determinant factors of AC systems. Monte Carlo simulation also allows the input variables that are not independent of each other to be modelled and incorporated into equations 1 and 2. Since there is a lack of data, there are two probability distributions that are suitable for modelling input variables in the absence of sufficient information or when the system to be modelled does not exist. These two probability distributions are the triangular and the beta distributions (Law and Kelton 1991, Back, Boles and Fry 2000).

The capital cost of the four AC systems is based on the actual data reported by (Cassidy 2000). The cost of running the four AC systems is based on assumptions reported in the same publication. These values can be replaced by more accurate data if and when better information is available to the analyst. The model used to represent input data follows the three point estimating procedure (triangular distribution function). This method requires the analyst to set a lower and an upper limit to a most likely base estimate for every cost item for each AC systems under investigation, according to the most optimistic and pessimistic conditions, respectively. The degree of variability is indicative of the level of certainty about the cost under investigation. Monte Carlo simulation can be applied to determine the distributions of NPV, given that the probability distribution of each variable is known. The probability distributions of the AC determinant parameter can be developed from experience of similar occupancy costs in approximately similar conditions. The triangular distribution was selected for modelling the occupancy cost input variables. The rational behind the selection of triangular distribution is discussed in (Fente et. al 2000, Law et al. 1991 and Wall 1997, Chau 1995):

**Table 1: Input Parameters**

| **Air Conditioning System** | **AC installation costs £/m2** | **Electrical work costs (£/m2)** | **Other associated costs (£/m2)** | **Every 15 yeas major replacement asset (£/m2)** | **Every 5 years sub-replacement asset (£/m2)** | **Annual planned preventative maintenance (£/m2)** | **Annual Energy costs (£/m2)** |
|---|---|---|---|---|---|---|---|
| Variable air Volume with perimeter heating | 228 | 7.5 | 96 | 130 | 42 | 62 | 219 |
| Variable refrigerant flow | 166 | 6 | 66 | 212 | 55 | 123 | 136 |
| Fan-coil units | 183 | 7 | 72 | 89 | 62 | 109 | 137 |
| Chilled ceilings | 229 | 3 | 88 | 48 | 48 | 27 | 62 |

The parameters of the triangular probability-density function can be estimated using expert subjective judgement (Chau 1995) or from historical data using moment matching, maximum likelihood estimation, and least-squares fit of the cumulative distribution function (AbouRizk et al. 1994).

If an input variable has a high uncertainty the coefficient of variance of this variable can increase depending on the perceived uncertainty associated with the variable. The most likely values in Table 1 were extracted from actual cases. These values along with the assumptions made about the upper and lower limits of each variable in Equation (1) were used to estimate the parameters of the probability distribution function of the input variables. Goodness-of-fit tests were used to select the best probability distribution function that represents NPV output variables. Monte Carlo simulation is used to generate the distribution of possible NPVs. It takes samples from the input variable distributions and evaluates the corresponding NPV that is a function of these variables. The process is repeated for 2000 iterations and the resulting $NPV_1$, $NPV_2$,….$NPV_n$ are used to obtain the cumulative distribution of NPV. Obtaining NPV values in this way is subject to estimation error resulting from sampling error, and inappropriate discount and inflation rates. Therefore, the reliability of the generated distribution has to be assessed. In this work the Anderson Darling goodness of fit test, which checks if the observed data could have originated from a theoretical distribution with the estimated parameters.

## 4 RESULTS AND DISCUSSION

Table 2 shows the results of matching the forecasted NPV of each AC system to a theoretical distribution function. The matching was based on the Anderson-Darling goodness-of-fit test. A total of 32 continuous distributions were assessed. Those, which best represent, the forecast data along with their characteristics are shown in Table 2. Table 2 shows that the Gamma distribution is ranked first for the VAV AC system. But the Random Walk distribution was found to be the best fit for the VRF Ac system. It was found that the fan coil unit and chilled ceiling Ac systems is best represented by the Johnson SB distribution.

Figs 1-4 Illustrate the preliminary analysis carried out to check for the goodness-of-fit of the selected distribution and the resulting distribution-function-differences (DFD) plot for the forecasted NPV values for the investigated AC systems. The DFD plots show that the vertical differences between the sample distribution and the theoretical distribution are close to zero, which indicates that, the selected distribution for representing maintenance cost is a good match. Based on the above evidence and deduction, it appears that the selected distributions provide a good representation of the NPV values.

**Table 2: Characteristics of the selected distributions for all AC systems**

| **Descriptives for VAV WLCC Model** | | | | |
|---|---|---|---|---|
| Model | Mean | Variance | Skewness | Kurtosis |
| 1. Sample | 3824.859 | 107226.2 | 0.14612 | 3.00017 |
| 2. Gamma | 3824.859 | 107312.9 | 0.17129 | 3.04401 |
| **Descriptives for VRF WLCC Model** | | | | |
| Model | Mean | Variance | Skewness | Kurtosis |
| 1. Sample | 3539.283 | 41990.82 | 0.1635 | 2.99374 |
| 2. Random Walk | 3539.283 | 41987.63 | 0.17349 | 3.05014 |
| **Descriptives for Fan Coiled WLCC Model** | | | | |
| Model | Mean | Variance | Skewness | Kurtosis |
| 1. Sample | 3250.953 | 39889.1 | 0.10615 | 2.77496 |
| 2. Johnson SB | 3252.941 | 39889 | 0.1 | -0.219 |
| **Descriptives for Chilled Ceiling WLCC Model** | | | | |
| Model | Mean | Variance | Skewness | Kurtosis |
| 1. Sample | 1507.425 | 9731.299 | 1.17E-03 | 2.64078 |
| 2. Johnson SB | 1506.844 | 9731 | 0.0012 | -0.355 |

To validate this deduction, a goodness of fit test is carried out to assess formally the quality of the selected distribution representation. The results of this test are shown in Table 3. Since the modified statistic test is significantly lower than the critical values for level of significance shown in Table 2, the selected distribution cannot be rejected at all alpha levels indicated in the table. Hence, there is no reason to believe, based on the above evidence and tests that the selected distribution does not provide a good fit for NPV values of each of the investigated AC systems. Fig. 2 shows NPV percentiles comparison of the floor AC systems. Using the median as a benchmark for comparison the VAV system is found to be more expensive to install and run over the life cycle, whereas the chilled ceiling system is found to be the most cost effective option to acquire and operate. The rational behind this can be attributed to factors that are discussed by Cassidy (2000)

**Table 3: Results from Anderson-Darling goodness of fit test**

| **WLCC Model** | **First ranked distribution** | **Anderson Darling Test Statistic** | **Critical Values for Level of Significance (alpha)** | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | **0.25** | **0.1** | **0.05** | **0.025** | **0.01** | **0.005** |
| VAV System | Gamma | 0.26092 | 0.47 | 0.63 | 0.752 | 0.873 | 1.04 | 1.159 |
| VRF System | Random Walk | 0.21682 | 1.248 | 1.93 | 2.492 | 3.07 | 3.86 | 4.5 |
| Fan coil system | Johnson SB | 0.305 | 1.248 | 1.93 | 2.492 | 3.07 | 3.86 | 4.5 |
| Chilled ceiling system | Johnson SB | 0.489 | 1.248 | 1.93 | 2.492 | 3.07 | 3.86 | 4.5 |

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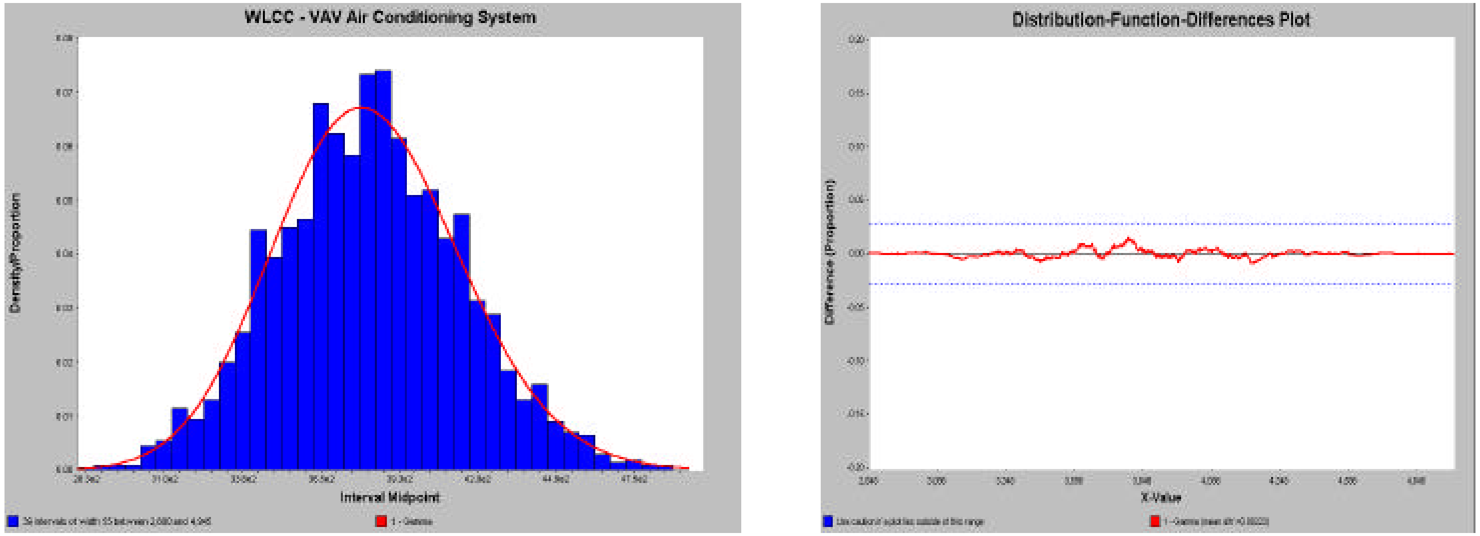


**Fig 1: Probability Distribution Function and Distribution Function Difference Plot for VAV system**

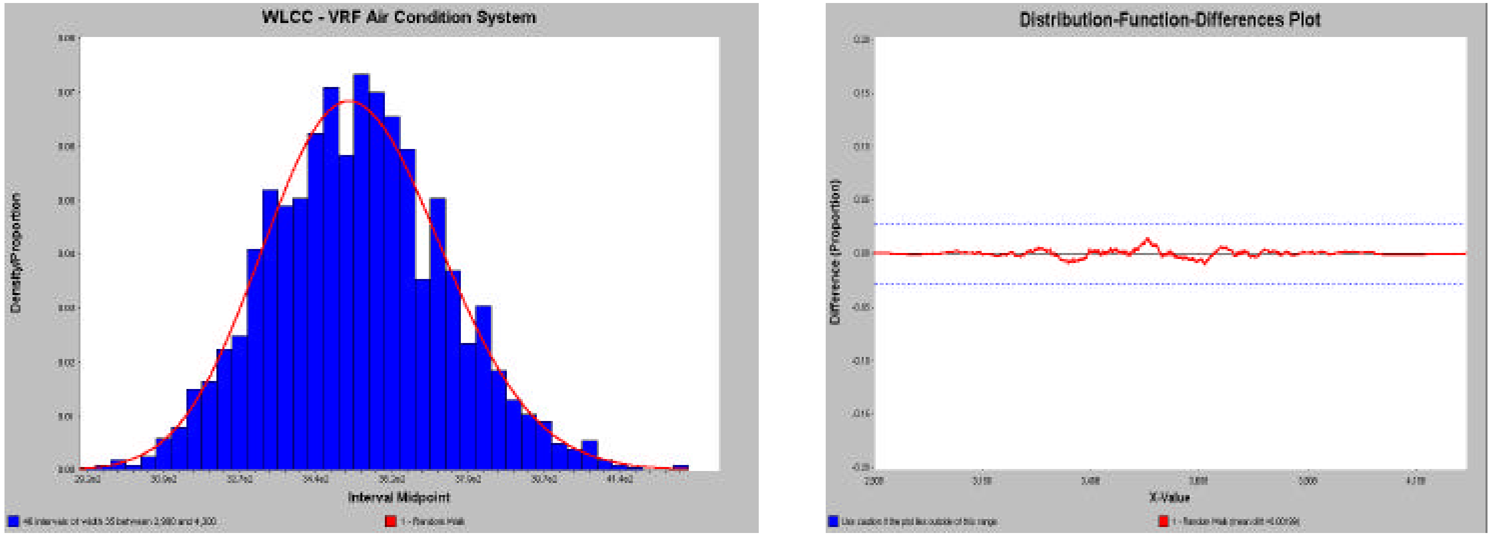


**Fig 2: Probability Distribution Function and Distribution Function Difference Plot for VRF system**

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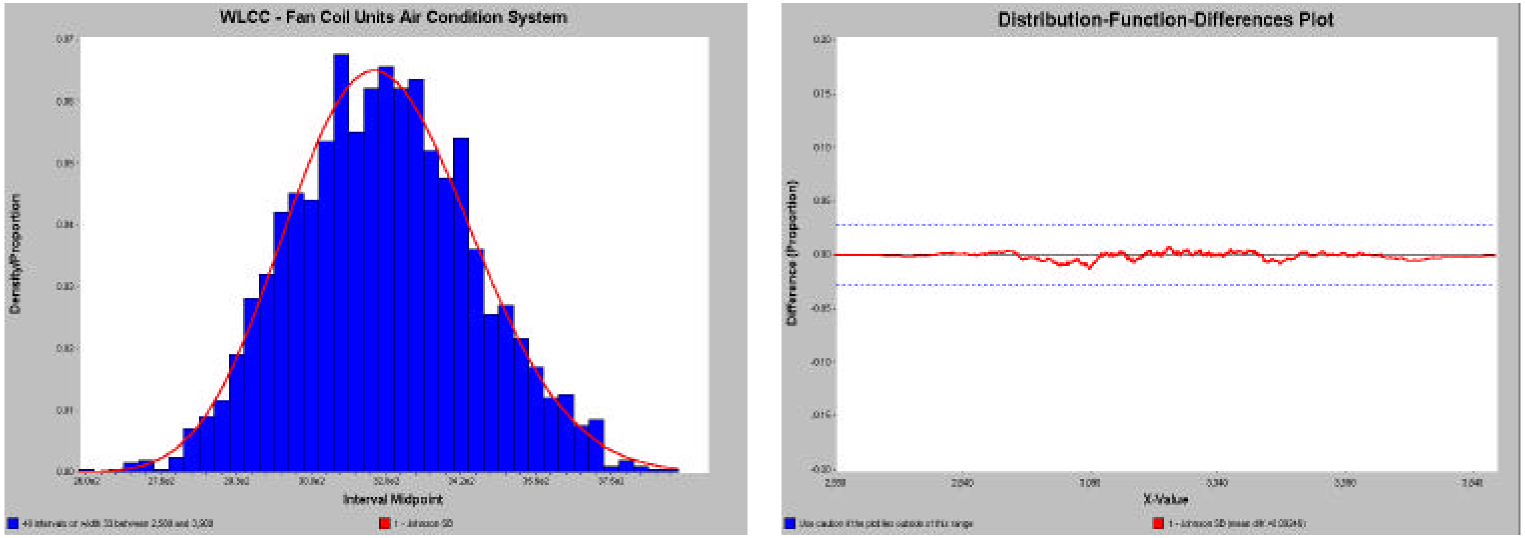


**Fig 3: Probability Distribution Function and Distribution Function Difference Plot for Fan Coil System**

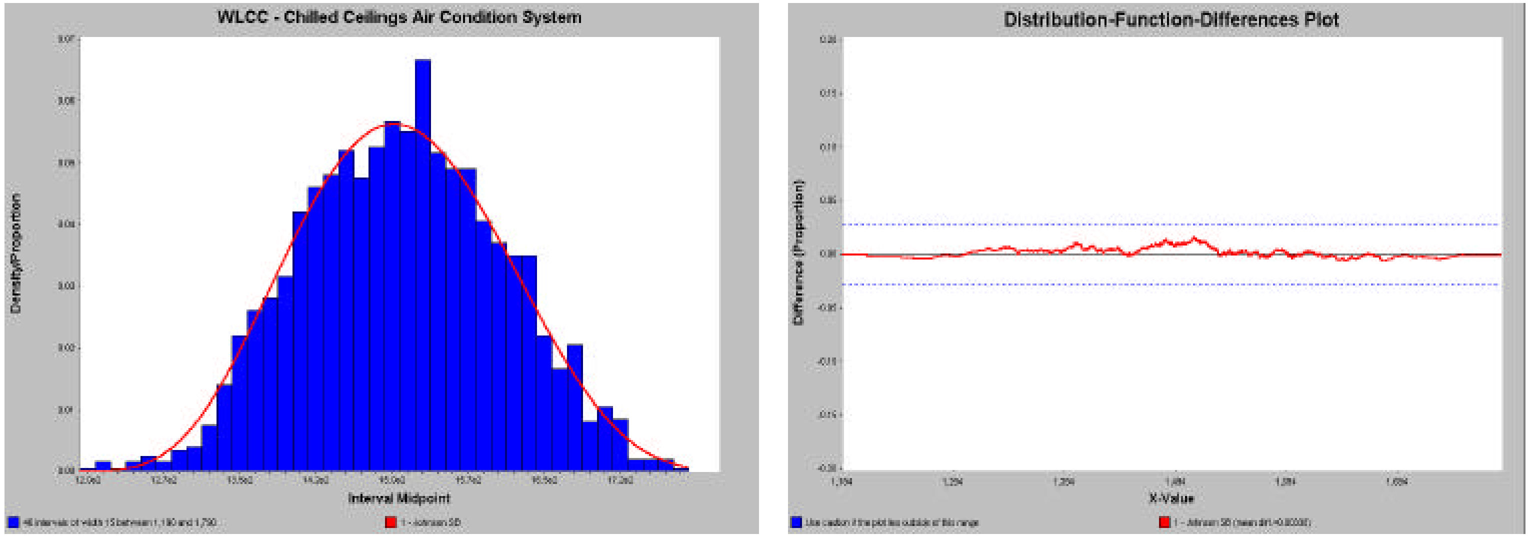


**Fig 4: Probability Distribution Function and Distribution Function Difference Plot for chilled ceilling**

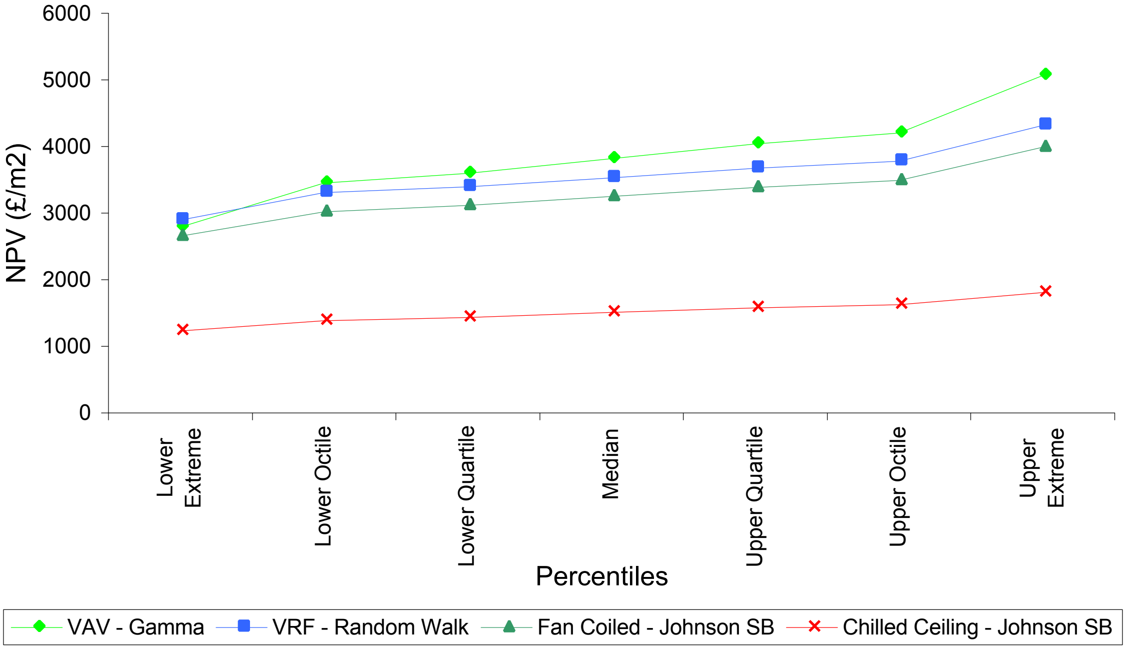


**Fig 5: NPV Percentiles Comparison Chart for all WLCC Models**

Assessing the economic viability of AC systems over their life expectancy is difficult to establish with certainty. Hence, it is very difficult to project future cash flows as single point cost forecasts. Monte Carlo simulation provides a tool, which assists cost analysts to take into consideration the variability, and uncertainty in the modelling of the input parameters. In this study the input parameters in equations 1 and 2 are considered random variables with triangular distributions (although other distributions could be assessed for their goodness of fit representation if data are sufficiently available). Monte Carlo simulation allows the entire distribution of NPV of each investigated AC systems to be analysed and informed decisions can be taken regarding the viability of each system. Hence, instead of computing a single value of NPV for each AC system under study, the simulation provides a probability distribution of NPV values for each system. The results generated from this simulation model presented NPV by equations 1 and 2 are the expected values and standard deviations of NPV. The NPV distribution of each AC system provide an indication to the economic viability of each AC system and the standard deviations provide information to decision makers about the variability or uncertainty associated which AC system.

## 5 CONCLUSION

This paper presented an initial investigation into the modelling whole life cycle cost of four air conditioning systems. The approach incorporated the notion that the whole life cost of these systems dependent on several random latent variables. Hence, the input variables to the model are treated as random quantities. The analysis shows that Gamma, Random Walk and Johnson SB distributions best represent the four investigated systems. The chilled ceiling and beam option is found to be cost-effective over the life span of the system. The results also might be useful for financial planning and budgeting for maintenance costs.

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# Between Sustainable And Durable: Optimization Of Life Spans

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**Summary: Durability and sustainability are often seen as two separate and sometimes even opposite approaches of building design and construction. Although durability and sustainability not always go together, both views have to be integrated for a sustainable built environment, as durability is part of sustainability. The aim of this paper is to determine the potential role of durability in sustainable houses. Calculations were made using the Dutch tool Eco-Quantum. A terraced house was applied as a reference. The influence of life spans of houses and house components and of recycling and reuse on the environmental impacts of houses was studied. It appears that durability provides a significant contribution to sustainability, but it competes with other strategies to reduce the environmental impacts of houses, such as energy saving, flexibility and use of renewable materials. Furthermore, allocation problems form an impediment for taking the environmental benefits of reuse and recycling into account in an appropriate way. Therefore, houses and house components with relatively short life spans, but good opportunities for recycling and reuse, are put at a disadvantage in comparison with houses or house components with relatively long life spans, but which do not meet these recycling and reuse properties. Further research is recommended to be able to optimize life spans.**

**Keywords: sustainable, durable, Eco-Quantum, environmental impacts, recycling and reuse**

## 1 INTRODUCTION

Durability and performance of building components and materials have to be assessed in a much broader context nowadays. Not only building components and materials are being increasingly assessed as part of a building and over the total life cycle today, but also sustainability is an important consideration. Some people interpret sustainability purely ecological, others as prolongation of life spans. The point is that these interpretations are not contradictions, but representations of two aspects of sustainable building. In other words, durability is an aspect of sustainability, although sustainable does not have to mean durable and vice versa. For example, the XX office in the Netherlands was designed to last twenty years at maximum (Klomp & Post 1999). After this period all materials have to be released, reused or recycled completely. This certainly is not durable, but it can very well be sustainable. The importance of integrating these approaches has already been emphasized (e.g. Duijvestein 2000, Hendriks 2000). This paper provides a more objective discussion. It aims at quantifying the potential role of durability in sustainable buildings and building components. It focuses on housing. The study is part of a PhD research about optimization of the environmental performance of houses.

This paper starts with an explanation of the concept of sustainable development and its relationship to the built environment. It also sketches the paradox between durable and sustainable. Next, the paper elaborates on environmental impact assessment. It describes the methodology of Life Cycle Assessment and the Dutch tool Eco-Quantum. Furthermore, a reference house is introduced for the calculations presented in this paper. After that, the influence of life spans of houses and house components on the total environmental impacts of houses is accounted and compared to the environmental benefits of other strategies to reduce these impacts. Consequently, the paper addresses the problem of allocation of recycling and reuse. This is expected to have a major influence on the question raised in this paper. Methods for allocation are explained and judged upon their consequences for durability and sustainability. Finally, conclusions will be drawn with respect to the importance of life span as a decisive factor for sustainable houses or, in other words, the question is dealt with whether to build for eternity or temporality.

## 2 LINKING SUSTAINABILITY AND DURABILITY

### 2.1 Sustainable development

The report our common future (World Commission on Environment and Development 1987) has led to a worldwide notion of the concept of sustainable development, which was defined as a “development which meets the needs of the present without compromising the ability of future generations to meet their own needs.” The commission not only observed environmental problems to be addressed, but also social problems, such as inequity, poverty, non-prosperity and violation of human rights, related to explosive population growth and enormous expansion of environmental harms caused by human activities. According to the commission, solving these problems asks for global economic growth while respecting ecological constraints.

The concept of sustainable development has been adopted by a growing number of (international) companies, known as the triple bottom line approach (Elkington 1998). The triple bottom line broadens the pursuit of profitability as the traditional bottom line with environmental and social added values. For that, the triple bottom line consists of society, economy and environment, also indicated as people, profit and planet. It conveys that society depends on the economy, whereas the economy depends on the global ecosystem. A healthy global ecosystem is the ultimate bottom line. Another approach is natural capitalism (Lovins *et al.* 1999), which puts emphasis on solving environmental problems by radically more productive use of natural resources.

Although it is clear that sustainability is not only about eco-efficiency and companies are able to work with it, it is difficult to handle the strategic concept of sustainable development with respect to operational decisions for a sustainable built environment. The ecological conditions strategy (Tjallingii 1996) offers more opportunities to do so. It does not focus on future results, but on present steps in a sustainable direction by providing guiding principles. Three dimensions of sustainability are distinguished: durable diversity of areas, sustained use of resources and sustained involvement of actors. These dimensions are indicated in short as areas, flows and actors.

Nonetheless, narrowing the scope from the built environment towards buildings and building components entails that sustainability is translated as responsible management of flows. After all, on the scale of the building and its components areas and actors are of minor relevance compared to the scale of the neighbourhood or the city. The definition of sustainable construction, according to the Dutch ministry of Housing, Spatial Planning and the Environment (1993), confirms this. The ministry explains sustainable construction as directed towards reduction of the environmental and health impacts as a consequence of construction, buildings and the built environment. Therefore, this paper deals with the perspective of flows. From this viewpoint, sustainable buildings are characterized by minimization of the environmental impacts of material use, energy consumption and water consumption during the entire life span of buildings.

### 2.2 Durable or sustainable?

Prolongation of life spans of buildings and building components is one of the possibilities to minimize the environmental impacts. In that case, durability comes up, which means appropriate to exist a long time, without unacceptable degradation of relevant functional characteristics (Hendriks 2000). It is obvious that a building with a life span of seventy-five years causes less environmental impacts than the same building with a life span of fifty years, assuming that the latter will be replaced by a new one. However, this is not that obvious when comparing two different buildings. In that case, it does not have to be that the building with a life span of fifty years causes more environmental impacts than the building with a life span of seventy-five years. That strongly depends on the materials used and the energy and water consumption. The same reasoning applies to building components, although energy and water consumption is left out of consideration at that level. Durable building components with a relatively long life span certainly can bring about more environmental impacts than sustainable building components with a relatively short life span. For example, a building component with a life span of forty years with an environmental burden, which is three times higher than a building component with a life span of twenty years with a three times lower environmental burden, ultimately causes more environmental impacts. Within forty years, the latter has to be replaced exactly once, so in the end the environmental burden is one third less than the first.

As well as striving for prolongation of life spans of the same building or building component in the same appearance, striving for reuse and recycling of building components and materials is an option to extent life spans. This is sustainable by means of reduction of the environmental impacts during the whole chain of subsequent life spans, but it does not have to be durable. Building components and materials might last shorter, provided that these will be reused or recycled, without compromising the total environmental impacts. Recycling comprehend all processes in the entire cycle of building components and materials, from ‘new’ to ‘old’ and from ‘old’ to ‘new’, in which the latter stands for secondary materials or products. Unlike recycling, reuse implies that used building components and materials will be used again for the same purpose as before or for an alternative purpose, but at the least in the same function, without further processing other than upgrading (Hendriks *et al.* 1999b). Often durable materials, such as metals, cause relatively large environmental impacts, while these are more suitable for recycling. This complicates deliberations about durability and sustainability. Moreover, buildings have relatively long life spans, which makes it very difficult to predict future reuse and recycling scenarios and innovative construction methods, such as IFD (industrial, flexible and dismountable construction), raising the potential for reuse and recycling.

It is certainly not either durable or sustainable, but it is important to compromise both of them. Duijvestein (2000) has catched this in Fig. 1. It is essential to consider the entire life cycle to be able to find out the overlap between durable and sustainable.

To what extent this overlap exists, is largely unknown. Life Cycle Assessment, or LCA, offers a framework to quantify the match between durable and sustainable.

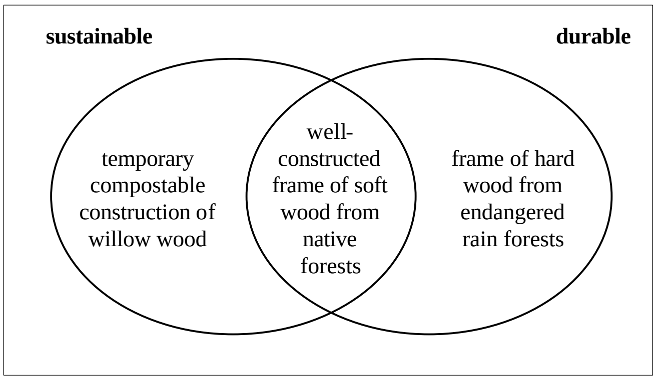


**Figure 1. Between sustainable and durable**

# 3 ENVIRONMENTAL IMPACT ASSESSMENT

## 3.1 Life Cycle Assessment

LCA is a method for the analysis of the environmental burden of products (goods and services) from cradle to grave, including extraction of raw materials, production of materials, product parts and products and discard, either by recycling, reuse or final disposal (Guinée *et al.* 2001). It is defined as the "compilation and evaluation of the inputs, outputs and potential environmental impacts of a product system throughout its life cycle" (ISO 1997). The product system is the total system of processes needed for the product, which in this case is a house. Inputs and outputs are materials and energy, which enter respectively leave the product system.

The framework for LCA, which has been internationally agreed upon, distinguishes four phases, as is shown in Fig. 2 (ISO 1997).

- Goal and scope of an LCA have to be clearly defined and geared to the intended use. An important part of the goal and scope definition is determination of the functional unit, which is the quantified function of the product system under study, to serve as reference unit in an LCA, e.g. x $m^2$ floor system with a supporting power of x $N/m^2$ during x years.
- Inventory analysis is the second phase of an LCA, in which the inputs and outputs of the product system are compiled and quantified, including natural resources and emissions to air, water and soil.
- The third phase is concerned with understanding and evaluation of the magnitude and significance of the potential environmental impacts of the product system. Impact assessment encompasses assignment of inventory data to impact categories (classification), modeling of inventory data within impact categories (characterization) and, only if useful, aggregation of the results (weighting). Examples of impact categories are depletion of natural resources and ozone depletion, acidification and eutrophication.
- Finally, the interpretation phase contains interpretation of the results of the inventory analysis and impact assessment in the light of the goal and scope definition in order to draw up conclusions and recommendations by means of completeness, sensitivity, consistency and other checks (ISO 1997).

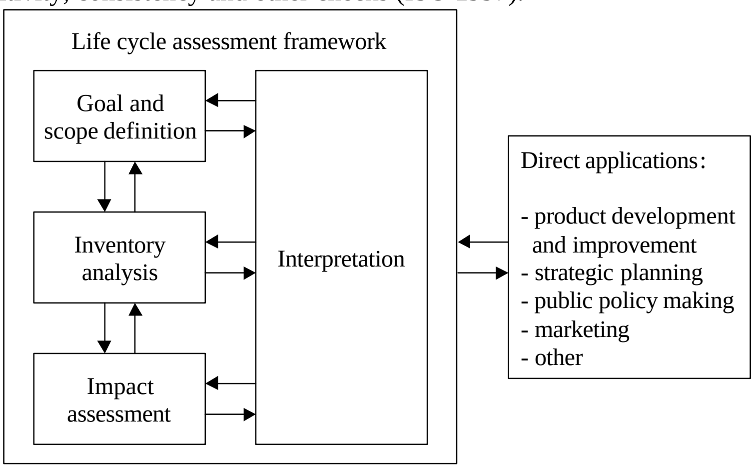


**Figure 2. Phases of LCA**

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### 3.2 Eco-Quantum

From international projects, for example, within the International Energy Agency (Knapen & Boonstra 1999) and the Green Building Challenge (Cole & Larsson 2000), it appears that the Netherlands goes ahead concerning the development of environmental assessment tools. Therefore, the Dutch tool Eco-Quantum was used to make the calculations presented in this paper (Mak *et al.* 1999).

Eco-Quantum has the following features.

- It is a tool for environmental impact assessment of houses, meant for architects, clients and municipal councils who can use it for, amongst others, optimizing designs, benchmarking and policy framing.
- It conducts an LCA of the flows of materials, energy and water in houses. The flow of materials regards the use of materials of all house components, including material-embodied energy. The flows of energy and water comprise energy consumption and water consumption respectively in the user phase of the house.
- Twelve impact categories can be analyzed with Eco-Quantum: depletion of raw materials, depletion of fuels, global warming, ozone depletion, acidification, eutrophication, human toxicity, ecological toxicity, photo-oxidant formation, energy consumption, non-hazardous waste and hazardous waste. The latter three are not really impact categories, but pressure indicators (MegaJoules and kilograms). Although such pressure indicators have a strongly recognizable function, this paper sets them aside to avoid double counting.
- **Reference house**

A reference house was chosen to make the calculations. Three types of houses represent current Dutch housing construction: the terraced house, the semi-detached house and the gallery apartment (Novem 1999). The terraced house (see Fig. 3) was set as a reference house for this study, as the most common and wanted house in the Netherlands. It has four rooms and three storeys. It holds a saddle roof and a fixed stair leading to an attic. The living room and the open-plan kitchen are on the ground floor. There are three bedrooms and a shower on the first floor. The usable floor area of the house is 111 $m^2$; the gross volume is 352 $m^3$. Table 1 gives an overview of the most important construction and installation characteristics of the house.

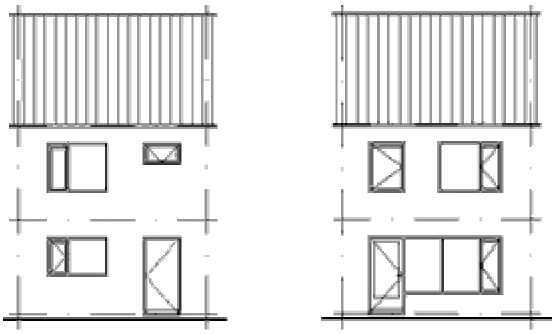

**Figure 3. Drawing of the reference house**

To identify the potential role of life spans of houses and house components on the total environmental impacts, the flows of materials, energy and water of the reference house were taken into account, including replacements and maintenance, but excluding renovations and refurbishments. Dutch houses have an average life span of seventy-five years, so this was assumed as standard life span. Figure 4 shows the distribution of the environmental impacts among material use, energy consumption and water consumption of the reference house. It turns out that the environmental impacts of material use amounts more than a half on depletion of raw materials, ozone depletion, photo-oxidant formation, human toxicity, ecological toxicity, acidification and euthrophication. Furthermore, the environmental impacts of energy consumption amounts more than a half on depletion of fuels and global warming. Finally, the environmental impacts of water consumption are of importance on ozone depletion only.

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**Table 1. Construction and installation characteristics of the reference house**

| *House component* | *Characteristics* |
|---|---|
| Foundation | concrete beams and piles |
| Fronts | inner leave of sand-lime brick, rockwool cavity insulation and outer leave of masonry ($R_c$=3,0 $m^2K/W$), frames, windows (U=1,7 $W/m^2K$) and doors (U=3,4 $W/m^2K$) of unstamped sustainable wood[1] |
| Parting walls | sand-lime brick |
| Interior walls | plaster |
| Floors | ground floor of ribbed waffle slabs fitted with EPS insulation ($R_c$=3,0 $m^2K/W$) and storey floors of hollow-core slabs |
| Roof | wooden roof elements clad with unstamped multiplex, insulated with EPS plates and covered with concrete tiles ($R_c$=3,0 $m^2K/W$) |
| Heating and hot water | low-$NO_x$ combined boiler |
| Ventilation | mechanical ventilation system |

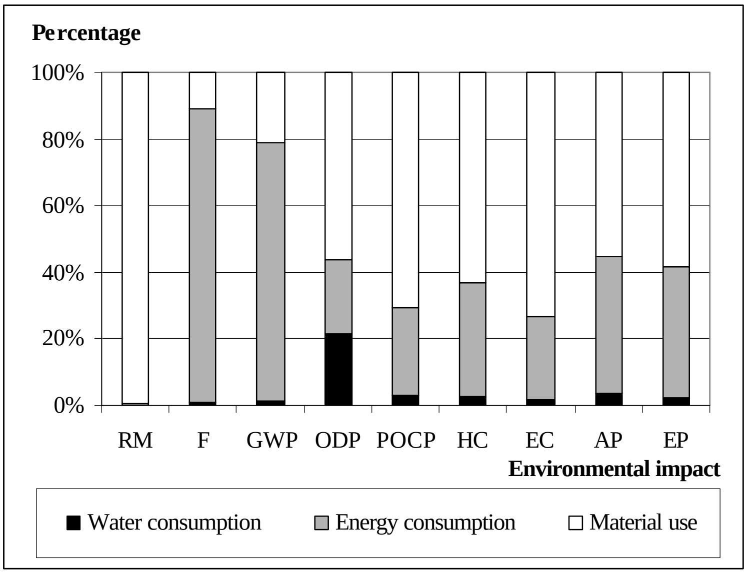


**Figure 4. Distribution of environmental impacts among material use, energy consumption and water consumption**

Explanation of signs: RM depletion of raw materials, F depletion of fuels, GWP global warming potential, ODP ozone-depletion potential, POCP photo-oxidant formation, HC human toxicity, EC ecological toxicity, AP acidification potential, EP eutrophication potential.

## 4 ROLE OF LIFE SPANS

### 4.1 Life spans of houses

4.1.1 Increasing life spans go together with decreasing environmental benefits

The longer the life span of a house, the higher the environmental impacts of materials, energy and water are, due to replacements and maintenance of house components and energy and water consumption during the user phase of the house. Nonetheless, the environmental impacts per year decrease, because some house components only cause environmental impacts during construction. For instance, the load-bearing structure needs neither replacements nor maintenance in the user phase, so the environmental impacts of construction are the same for all life spans, whether it is 30 or 120 years. Apart from that, there

1 To be understood as a sustainability category, and not to be confused with sustainably produced wood. In the latter case we speak of stamped wood.

are some house components, which have a remaining life span if the life span of the house is shorter than the life span of the house components. In that case, the environmental impacts per year of a house increase, because these are provoked for a shorter period than the technical life span. Therefore, houses with relatively long life spans yields environmental benefits, while houses with relatively short life spans have to be rebuild once or more in the same period. However, these environmental benefits diminish, because more and more house components need replacement and maintenance anyway. This trend is shown in Fig. 5 for seven different life spans of houses with respect to depletion of raw materials. The horizontal axis shows the years. The vertical axis contains the indexes of the scores on depletion of raw materials, in which the initial environmental burden during construction was set at 100. Over a long period, i.e. thousands of years, the environmental burden of the houses with relatively short life spans is considerably higher than the other ones. However, as life spans increase, differences in environmental burden decrease. At 75 years, the house with a life span of 120 years has a negligible lower burden and the house with a life span of 30 years a 40% higher environmental burden on depletion of raw materials than the house with a life span of 75 years. The same orders of magnitude apply to the other environmental impacts.

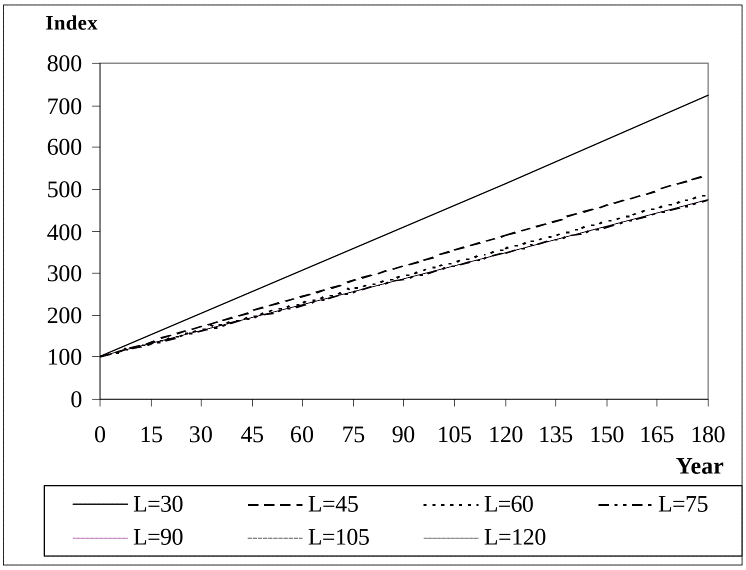


**Figure 5. Trends in indexes of scores on depletion of raw materials during the years for different life spans of houses**

4.1.2 Increasing life spans not always mean decreasing environmental impacts

Over a very long period there is a matter of unambiguous raise of environmental benefits, as life spans become longer. Yet, zooming in on the actual path of environmental impacts during the period from 0 to 180 years demonstrates a more complex image. It can very well be that in a given year in this period the environmental impacts of a house with a longer life span is higher than the environmental impacts of a house with a shorter life span. This is caused by different phases related to construction of new houses at the end of different life spans, as exemplified by Fig. 6. In this figure the indexes of scores on depletion of raw materials during the years are reproduced for life spans of 30, 40, 75 and 90 years. The straight-up lines represent construction of new houses. The inclining lines represent user phases. Actually, these should not be linear lines, because replacements and maintenance should cause peaks at certain years. However, Eco-Quantum accounts last times replacements in proportion to the remaining life span of the house. Therefore, it is not really justified to compare the houses at any given year. For example, at 180 years the house with a life span of 75 years scores higher on depletion of raw materials than the house with a life span of 45 years. However, the latter then has to be built again, while the first has not even reached half of its life span. In the case of environmental impacts on which energy and water consumption has a very large share (see Fig. 4), it appears that the environmental burden features an almost linear raise during the years. In that case, houses with relatively short life spans distance itself fairly soon from houses with relatively long life spans, irrespective of the reviewed period. This means that house components, which have to be replaced relatively soon, prevail with respect to the environmental burden.

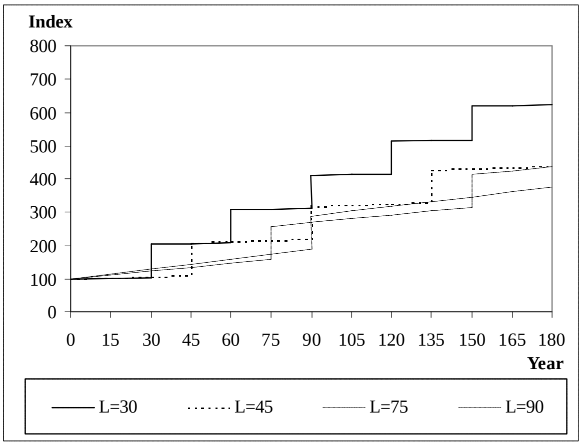


**Figure 6. Indexes of scores on depletion of raw materials during the years for different life spans of houses**

## 4.2 Life spans of house components

### 4.2.1 Increasing life spans go together with decreasing environmental benefits

Besides life spans of houses, life spans of house components could have a significant influence on the environmental impacts of houses. Therefore, life spans of a number of house components were varied, namely the outer leave, the window frames, the glazing, the rainproofing, the interior walls, the combined boiler for heating and hot water, the floor coverings and the roof construction. These are components, which exert considerable influence on the environmental impacts of houses (Klunder 2000). Load-bearing components were left out of consideration, because their life spans equal the life spans of the houses. The life spans of all components were decreased and increased by 5, 10 and 15 years. Because of the relatively short life spans of the glazing and the combined boiler, these components were only varied between -10 and +10 and -5 and +5 years respectively. Fig. 7 shows on the horizontal axis the alterations of life spans of house components and on the vertical axis the indexes of scores on all environmental impacts of a house with a life span of 75 years. The reference, i.e. an alteration of 0, was set at 100. It can be seen that fluctuations differ enormously. Depletion of raw materials shows the largest variations, whereas depletion of fuels shows the smallest variations. Moreover, shorter life spans of house components have more consequences than longer life spans. The environmental benefits of an increase of the life spans of important house components by 15 years in a house with a life span of 75 years amount to an average of 4% over all environmental impacts. The environmental drawbacks of a decrease of the life spans of house components by 15 years amount to an average of 9%. The highest raise is 33%, while the highest decline is 16%.

### 4.2.2 Increasing life spans of house components are particularly useful in houses with long life spans

Figure 8 displays the indexes of the scores on depletion of raw materials for seven different life spans of houses. It appears that the alterations of life spans entail minor differences in environmental burden with respect to houses with relatively short life spans. In that case, longer life spans of house components are of no importance, because the life span of the house is shorter for almost all house components then. The longer the life spans of the houses, the lesser are differences between the environmental consequences of alterations of the life spans of the house components.

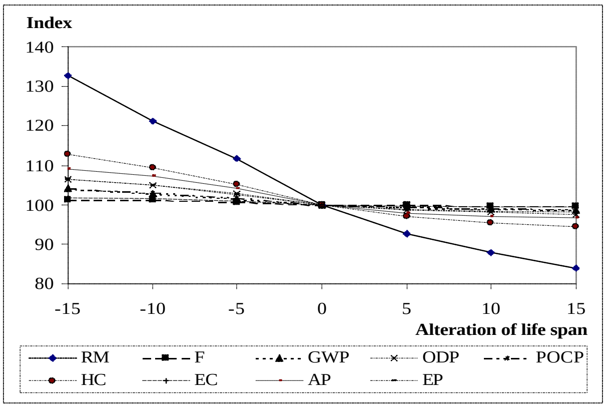


**Figure 7. Indexes of scores on all environmental impacts for different alterations of life spans of house components**

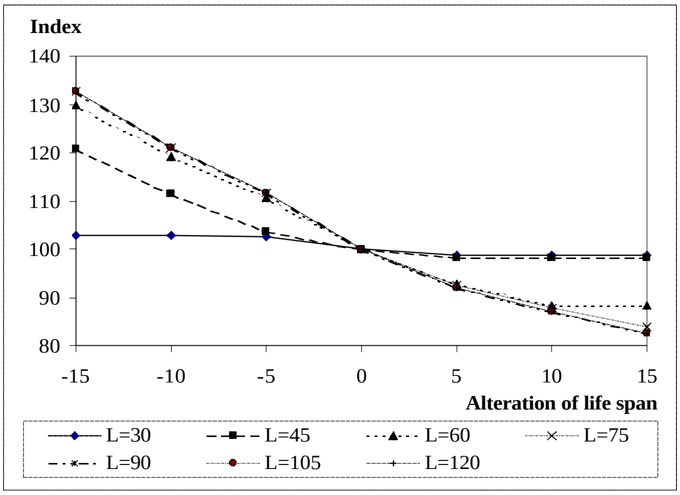


**Figure 8. Indexes of scores on depletion of raw materials for different alterations of life spans of house components and for different life spans of houses**

### 4.3 Environmental benefits of durability against other strategies

Durability is just one of the strategies to improve the environmental performance of houses. Other strategies are, for example, energy saving and renewable energy, dematerialization, use of renewable materials, use of less environmentally damaging materials, reuse and decrease in maintenance (Blaauw, 2001). Klunder (2001) accounted the environmental benefits of some measures, which belong to three themes: energy saving, flexibility and use of renewable materials. The order of magnitude of environmental benefits, as average percentages over all environmental impacts, vary between a raise by 10% (regarding energy saving by means of installation technology), a decline by 5% (with respect to energy saving through constructional measures and flexibility) and a reduction by 15% (pertaining to use of renewable materials). However, this fluctuates tremendously per environmental impact, namely from a raise by a factor of 2 to a decline by a factor of 5.

## 5 ROLE OF RECYCLING AND REUSE

Prolongation of life spans of house components and materials can also be realized through recycling and reuse. It is generally known that life spans of houses and house components as well as recycling and reuse have major consequences for the environmental impacts of houses. However, the environmental benefits of recycling and reuse are very hard to quantify for the time being. As yet, an unresolved difficulty in LCA is allocation. Often it is impossible to link processes in the product system by single material and energy flows. Most processes yield multiple inputs and outputs. These arise out of co-production of a number of products, combined waste treatment processes, production of recyclable material and use of recycled material. Therefore, inputs and outputs as well as associated environmental releases should be subdivided or allocated to the different products (ISO 1998, Guinée *et al.* 2001). Several procedures are applicable for this purpose, such as partitioning on the basis of physical relationships, e.g. mass, or economic value, e.g. scrap value.

Dealing with allocation of recycling and reuse in LCA is even more complex.

- First of all, recycling and reuse may imply that the inputs and outputs associated with unit processes for extraction and processing of raw materials and final disposal of products are to be shared by more than one product system.
- Secondly, recycling and reuse may change the inherent properties in subsequent uses. In open loops, which means that building components and materials end up with transformed properties in another product system, the number of subsequent uses of the recycled material belongs to the allocation parameters, besides physical properties and economic value (ISO 1998). For instance, the environmental benefits of recycling and reuse of house components and materials can be ascribed to the house from where these materials come from, but also to the house in where these materials will be used again. Besides, house components and materials can be recycled in product systems with lower-grade properties, such as concrete granulates in road constructions. Finally, upgrading and recycling processes also cause environmental impacts, which diminish the environmental benefits.

Hendriks *et al.* (1999a) distinguishes the following four kinds of allocation procedures.

1. Cut-off method: the environmental impacts of all processes in the product system are allocated to product A, with the exception of the environmental impacts of upgrading processes, which are allocated to product B.
2. Co-production and upgrading method: the environmental impacts of a whole product system are distributed over product A and B in case of co-production, but in case of upgrading only the upgrading processes are distributed over product A and B.
3. Subtraction method: the environmental benefits of recycling are allocated to product A by means of subtraction of a process, which would have taken place otherwise.

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4. Environmental impacts of primary production: the environmental impacts of primary production are distributed over product A and product B on the basis of the extent of recycling and quality losses.

Eco-Quantum handles allocation according to the most simple, but rough, cut-off method. The basic principle behind this choice is that no allocation of environmental benefits takes place now, while these benefits have to be realized in the distant future (Mak *et al.* 1999). Nevertheless, it underestimates the environmental benefits of recycling and reuse, because for the use of secondary materials and products general figures are applied, in which no special design methods, such as design for recycling or design for disassembly, are taken into account. Durability is better off in this context than sustainability, because not the whole life cycle is considered. It is of great importance to gain more insight in the environmental benefits of recycling and reuse, notwithstanding allocation problems. Thormark (2000) subscribes to the viewpoint that future environmental benefits should not be allocated to the original product, like the subtraction method, but establishes that due to allocation problems the phases after demolition of buildings are either excluded or handled in a rather limited way. Therefore, Thormark argues for considering the potential of recycling next to the total environmental impacts. This definitely is an interesting approach to reflect the environmental benefits of recycling and reuse, as well as to compare different scenarios, so future conditions and uncertainties can be assessed. This kind of information is indispensable for optimization of life spans.

## 6 CONCLUSIONS AND RECOMMENDATIONS

This paper addressed two fields of tensions between durable and sustainable.

1. Houses, house components and materials with long life spans, but with high environmental burdens, against houses, house components and materials with short life spans, but with low environmental burdens.
2. Houses, house components and materials with high environmental burdens, but suitable for recycling and reuse, against houses, house components and materials with low environmental burdens, but unsuitable for recycling and reuse.

Concerning the first field of tension, this paper presented calculations on the influence of life spans of houses and house components on the environmental impacts of houses, making use of Eco-Quantum and comparing to a reference house. This led to three main findings.

- Prolongation of life spans yields incontestable environmental benefits, because the environmental burden diminishes on all environmental impacts. Thus, a house with a life span of 30 years scores 33% worse and a house with a life span of 120 years negligible better than an house with a life span of 75 years. However, in a certain period, this has not to be this way, because differences in phases occur with respect to new construction of houses, which produces a leap upwards of the environmental impacts.
- Furthermore, prolongation of life spans with 15 years of some important house components leads to a decrease of environmental impacts by 4% with regard to a house with a life span of 75 years, while shortening in the same way leads to an increase of environmental impacts by 9%. Longer life spans of house components are not useful in houses with relatively short life spans. With regard to houses with relatively long life spans the environmental benefits of alterations of life spans of house components are almost the same for every house, whether these have a life span of 75 years or 120 years.
- The results on other strategies, such as energy saving, flexibility and use of renewable materials are much more ambiguous due to improvements on some environmental impacts, while others are getting worse. The investigated strategies result in orders of magnitude of environmental benefits between 5 and 15%, although in one case studied a drawback by 10% was observed. Nevertheless, these figures differ greatly per environmental impact.

This results in the conclusion that durability is not a decisive factor in sustainability, although it remains an important aspect of it. Although a Dutch terraced house was used as a reference, this conclusion is expected to be generally valid.

Regarding the second field of tension, this paper discussed the problem of allocation of recycling and reuse, which obstructs gaining knowledge on the environmental benefits of recycling and reuse. The cut-off method in Eco-Quantum disadvantages designs, which anticipate on future recycling and reuse. Therefore, the environmental benefits of durable houses and house components should be seen in a broader perspective. Until now, methodological difficulties in LCA overshadow discussions on the potential of recycling and reuse, while this can put a much more positive view on houses with relatively short life spans.

Further investigation on the items mentioned above is needed to answer the question whether to build for eternity or temporality. Within this framework it should be regarded to what extent the XX philosophy offers starting-points in housing. The XX philosophy is based on the continuing shortening of economic life spans of office buildings in particular. Nonetheless, also construction of houses has to contend with heightening consumer’s wishes and fastening technological developments. Therefore, maybe this thought will provoke an impulse for innovative house designs, in which the right balance between durable and sustainable will be found.

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# Energetic And Environmental Pay-Back Time Of Solar Thermal Collector: A Scenario Analyses In An Italian Context

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**Summary: By using LCA method, each product is analyzed to discover the extent of its environmental impact. The paper focuses the attention on a thermal solar system for sanitary warm water demand. The main objective is to trace an eco-profile of this renewable technology, with regard to an exemplary plant, and secondarily to define energy and emissions payback time calculation. The methodology employed is "Cumulative Energy Demand" (CED) and values are obtained considering an averaged specific CED value. This plant was considered as belonging to different scenarios. In this way, we have had a general sight of how some parameters (range of CED values, range of specific emissions, collector inclination, user demand, tank volume, water flow, insulation thickness, surface of collector) could influence energetic and environmental payback time of the system. Analysis of results obtained from different scenarios can constitute a useful support for eco-design of solar thermal collector.**

**Keywords: Renewable energy, solar thermal collector, cumulative energy demand, life cycle analysis.**

## 1 INTRODUCTION

The general aim of this work was to investigate one of the renewable systems (RS), the solar thermal collectors, and make a comparison between the accumulated energy consumption of exemplary equipment with the energy harvested (EH) during their lifetime. A complementary objective is to make a balance between the pollutants released during the production processes (with particular care for the gaseous release) and the pollutants "saved" employing the RS. The first step was the calculation of the energetic expenditure related to one collector, considering the production phases from the raw materials to the finished product. This quantity is expressed by an index, the Cumulative Energy Demand (CED), that is the energy demand "valued as primary energy, which arises in connection with the production, use and disposal of an economic good" [VDI Richlinie 4600, 1997]. The calculation of the CED was made employing a mass balance method. The target product was split into its components and their respective constituent materials. After the realisation of this checklist, all the mass quantities were multiplied for a specific consumption factor, taken from the scientific literature [BUWAL, 1998] [ENET, 1994] [GEMIS, 2000] [Bousted, 1997a] [Bousted, 1997b] [Bousted, 1997c], referred to the unit of mass of output product (MJ/Kg).

## 2 THE ENERGY PAYBACK TIME

In addition to CED it was estimated the EH in the system useful lifetime. CED and EH values were employed in the calculation of the Energy Payback Time ($E_{PT}$) [Spreng, 1998], defined as the time during which the system must work to harvest as much energy (considered as primary energy) as it required for its production and disposal. $E_{PT}$ was obtained as :

(1)
$$E_{PT} = \frac{E_{cumulative}}{E_{Conv} - E_{User}}$$

where:

$E_{Cumulative}$ = Cumulative Energy for the production ($CED_P$) and disposal ($CED_D$) of the system [MJ];
$E_{Conv.}$ = Yearly Energy required by the conventional system substituted by the RS [MJ/year];
$E_{Use}$ = Yearly Energy necessary for the use of the RS [MJ/year].
These last two values can be calculated as follow:

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(2) $$E_{Conv} = q_{useful} \frac{A_C}{h_{Conv}}$$

(3) $$E_{Use} = e_{use} \frac{A_c}{h_{el}}$$

where:

$q_{useful}$ = Yearly harvested energy per unit of surface [MJ/(year*m$^2$ )];
$e_{Use}$ = Yearly energy demand for the use of the RS per unit of surface [MJ/(year*m$^2$ )];
$A_c$ = Collector surface [m$^2$];
$\eta_{Conv.}$ =Efficiency of a conventional system (it allows to obtain the energy save as Primary energy);
$\eta_{el.}$ = Overall efficiency of supply for power energy.
$E_{PT}$ is an important criterion can be used in a general framework of the ecodesign, locating the most energetic "expensive" materials used in a collector and searching for some more eco-compatible alternatives.

## 3 ENVIRONMENTAL LOAD AND EMISSION PAYBACK TIME

The study was focused mainly upon the gaseous release. These were calculated by employing specific factors that represent the average quantities of pollutants produced by the utilisation of one MJ of energy carrier. The emissions due to electricity were obtained considering the Italian electricity-mix [ANPA, 2000]. Emission Payback Time ($E_{M-PT}$) is defined as the time during which the avoided emissions are equal to the emissions necessary for production and working (assembly, use, maintenance) of the system. The index "i" is referred to the generic considered pollutant:

(4) $$E_{M-PTi} = \frac{EM_{cumulative,i}}{EM_{Conv,i} - EM_{Use,i}}$$

where :

$EM_{cumulative,i}$= Cumulative emissions of pollutant "i" released during the production, assembly and maintenance of the system [kg];
$EM_{Conv,i}$ = Yearly emissions of pollutant "i" that a conventional system should have produced when not substituted by the RS [kg/year];
$EM_{Use,i}$ = Yearly emission of pollutant "i" related to the use of auxiliary energy for the use of the RS [kg/year].

The Cumulative Emissions are obtained adding the emissions related to each employed material; these emission are estimated by multiplying each energy carrier mass, necessary for the process, for the specific emission factor. The other two components are obtained as:

(6) $$EM_{Conventional,i} = \frac{Q_{Conv} \cdot e_{Conv,i}}{h_{Conv}}$$

(7) $$EM_{Use,i} = E_{Use} \cdot ef_{Use,i}$$

where:

$Q_{Conv.}$ = Conventional energy substituted by the RS [MJ];
$ef_{Conv,i}$ = Specific emission factor for the pollutant "i" for the conventional system [kg/MJ];
$E_{Use}$ = Yearly Energy (power energy) necessary for the use of the RS [MJ/year];
$ef_{Use,I}$ = Specific emission factor for the pollutant "i" for the use of auxiliary energy [kg/MJ];
$\eta_{Conv.}$ =Efficiency of a conventional system

## 4 THE CED CALCULATION APPLIED TO A SOLAR THERMAL PLANT.

The concepts previously shown were applied to a case study. It is constituted by an exemplary water heating plant for sanitary demand of a single familiar house.

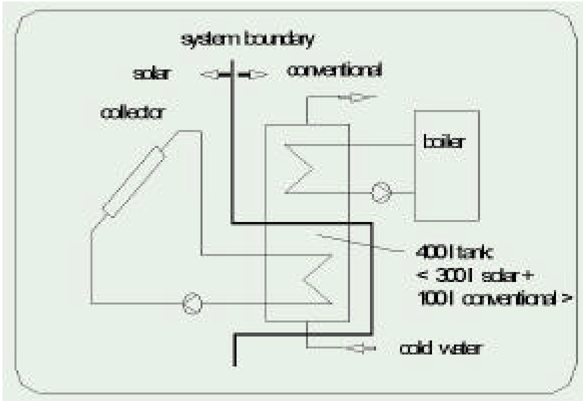


**Figure 1. Single-family-house system scheme.**

This plant includes: the collectors installed on the roof, pipes and connection, pumps, control system, water tanks and heat exchangers, see Fig 1. These components represent the "renewable plant" used as support to the conventional system. A conventional water heater (gas or electrical heater) has to be included in the plant in any case, employing or not the solar collector: the energy necessary for its production is so not included in the payback time calculations. The study was repeated for three different solar collector types: flat collectors (6.60 $m^2$), 3 collectors of "Vacuum 1" type (6 $m^2$) and 5 collectors of "Vacuum 2" type (5.25 $m^2$). The first was a flat collector that is the most common but also less efficient technology. The other studied collectors belongs to the vacuum-pipe technology. These collectors are constituted by a series of tubes inside whom vacuum is produced; in this way heat loss are reduced and the overall efficiency is increased. The two vacuum collector types differ themselves for the employed materials and the water flow inside them. The first collector (Vacuum 1) the heat is transferred to a medium and then to the water; in the second collector type (Vacuum 2) the water flows directly inside the pipes. The Fig.2 shows the results, considering the overall energy expenditure for the entire renewable plant (with different collector types) and the expenditure for each system component. It was so realised an Italian database about the specific CED-factors for all the material employed in the collector's production [Ardente, 2000]. In the Fig. 3 and Fig. 4 are reported the summaries about the overall main pollutants. The $CO_2$ was considered separately for its particular importance in the Global Warming effect.

## 5 "ENERGY SAVE" CALCULATION

The thermal performance of any type of solar thermal collector can be evaluated by an energy balance that determines the portion of the incoming radiation delivered as useful energy to the working fluid. For a flat–plate collector of an area Ac $_{\text{this}}$ energy balance on the absorber plate is:

(7) $$I_c \cdot A_c \cdot t_s \cdot a_s = q_u + q_{loss} + \frac{de_c}{dt}$$

where:

$I_c$ = solar irradiation on a collector surface [W/$m^2$];

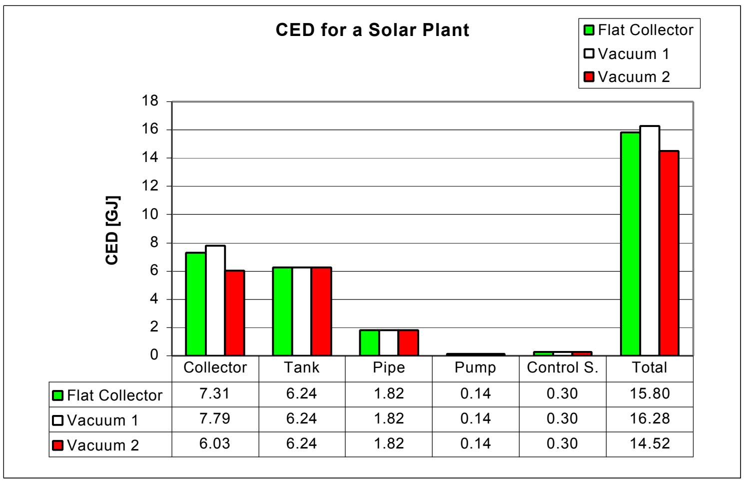


**Figure 2. CED for a solar plant in a single familiar house (system's components detail).**

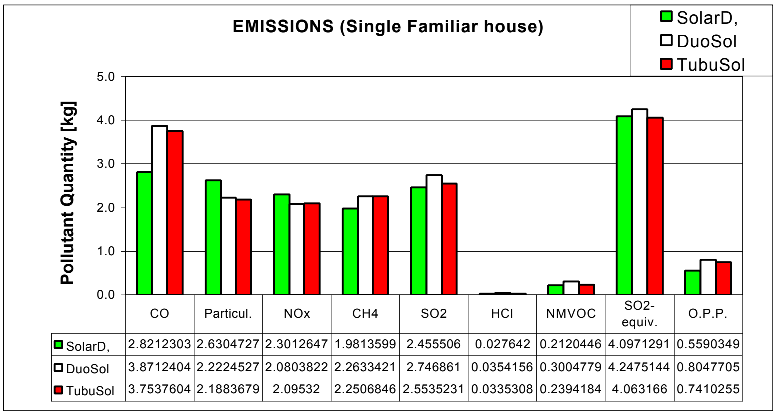


**Figure 3. Emission summary for the solar plant production.**

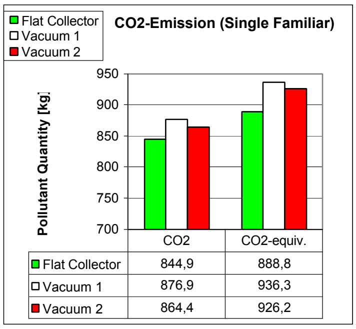


**Figure 4. $CO_2$ emissions for a solar plant production**

$A_c$ = collector surface [$m^2$];
$\tau_s$= effective solar transmittance of the collector cover (s);
$\alpha_s$= solar absorptance of the collector – absorber plate surface;
$q_u$= heat transfer rate from the collector absorber plate to the working fluid [W];
$q_{loss}$= rate of heat transfer (or heat loss) from the collector – absorber plate to the surroundings [W];
$d\,e_c /d\,t$ = rate of internal energy storage in the collector [W].
In order to construct a model suitable for a thermal analysis of a flat–plate collector, the following simplifying assumptions will be made [Duffie and Beckman, 1978]

- the collector is thermally in steady state;
- the temperature drop between the top and bottom of the absorber plate is negligible;
- heat flow is one-dimensional through the cover as well as through the back insulation;
- the headers connecting the tubes cover only a small area of the collector and provide uniform flow to the tubes;
- the sky can be treated as though it were a black-body source for infrared radiation at an equivalent sky temperature;
- the irradiation on the collector plate is uniform.

Once the collector heat–loss conductance $U_c$ is known, and when the collector plate is at an average temperature Tc and represents the ambient temperature, the collector heat loss can be written as:

(8) $$q_{loss} = U_c A_c (T_c - T_a) \text{ [W]}$$

The next step was the calculation of the useful heat $Q_u$ that represent the heat transferred to the fluid:

(9) $$Q_u = A_c \cdot F_R \cdot [S - U_c \cdot (T_{collector,in} - T_a)] \;\; [W]$$

where:

$F_R$ = removal factor that represents the heat rate transferred to the flowing fluid;
S = absorbed solar radiation [$W/m^2$];
$T_{collector,in}$= inlet fluid temperature [K];
$T_a$= Ambient temperature [K].

It was also possible to achieve the Useful energy transferred to the water tank, after knowing the efficiency of the heat exchanger:

(10) $$Q_{u,\tan k} = (G \cdot c_p) \cdot (T_{ex,in} - T_{ex,out})$$

where:

$T_{ex,in}$= temperature of the fluid at the entry of the heat exchanger;
$T_{ex,out}$= is the fluid temperature at the exit of the heat exchanger;
G = fluid flow [kg/s].
Where the temperature difference is obtained as:

(11) $$(T_{ex,in} - T_{ex,out}) = (T_{ex,in} - T_{\tan k}) \cdot \left(1 - e^{-\frac{U_{ex} \cdot A_{ex}}{G \cdot c_p}}\right)$$

and the symbols employed meaning :

$T_{tank}$ = is the tank temperature;
($U_{ex}$ *$A_{ex}$) = is the heat transfer coefficient for the exchanger [W/K];
$A_{ex}$ = exchanger percent surface.

The next part of the process is an estimation of the tank heat losses and the inner fluid flow caused by the user warm water demand. The user fluid flow is caused by supposing an user demand distribution during the day. The drawn heat is:

(12) $$E_{user} = c_p \cdot M_{user} \cdot (T_{\tan k} - T_{fluid})$$

where:

$M_{user}$= user fluid flow [kg];
$T_{fluid}$ = cold fluid temperature [K];
$T_{tank}$= tank temperature [K].

The temperature's values are not know a priori and they were so obtained by iterating the calculation. In table 1 are presented values of Useful energy ($Q_{useful}$), considering the three different collector types.

## 6 ENERGY PAYBACK TIME CALCULATION

The energy payback time was obtained employing the (1). It was necessary to obtain first the values of "$E_{USE}$" and "$E_{Conventional}$". The first term represents the electricity employed for the pump (neglecting the electricity employed by the control system) mainly constitutes it. The energy consumption was obtained as multiplication of the pump by the number of yearly working hours. The $E_{Conventional}$ was obtained dividing the overall energy save by the efficiency of the conventional system ($\eta_{conventional}$); in this way the end-energy save is converted in primary energy save. It was supposed that the renewable plant could substitute a conventional system with a gas or an electrical heater. The $Q_{useful}$ represents the end-energy save and it is the same in any case but the primary-energy saved is different in the two cases. The $\eta_{conventional}$ was obtained as inverse fraction of the conversion factors ($MJ_{Prim}/MJ_{End}$) considering the conventional energy carrier substituted by the solar energy . The same was made for the $\eta_{USE}$ (employed in the calculation of the "$E_{USE}$"). The results are summarised in Table 1 and 2.

**Table 1. $E_{USE}$ and $E_{conventional}$ employed for the Energy Payback time calculation.**

| | CED | $Q_{USE}$ | $\eta_{Electr.}$ | $E_{USE}$ | $Q_{useful}$ | $\eta_{Conventional}$ | | $E_{conventional}$ [GJ/year] | |
|---|---|---|---|---|---|---|---|---|---|
| | [$GJ_{Prim.}$] | [$GJ_{END}$/year] | | [GJ/year] | [GJ/year] | Gas | Electrical | Gas | Electrical |
| Flat collector | 15,80 | 0,676 | 0,312 | 2,17 | 9,94 | 0,943 | 0,312 | 10,53 | 31,85 |
| Vacuum 1 | 16,28 | 0,676 | 0,312 | 2,17 | 11,97 | 0,943 | 0,312 | 12,69 | 38,36 |
| Vacuum 2 | 14,52 | 0,676 | 0,312 | 2,17 | 11,76 | 0,943 | 0,312 | 12,47 | 37,69 |

**Table 2. Energy Payback Time for a Thermal Solar Plant in a single familiar house.**

| | Energy Payback Time [month] | |
|---|---|---|
| | Gas | Electrical |
| Flat collector | **22,7** | **6,4** |
| Vacuum 1 | **18,6** | **5,4** |
| Vacuum 2 | **16,9** | **4,9** |

Table 2 shows the relevance of the $\eta_{conventional}$. The payback time of a solar plant supported by an electrical heater is very little and it is always smaller than 1 year. In fact, to produce 1 MJ of power energy are necessary more than three MJ of primary energy. If solar plant substitutes an electrical heater, the energy save (or the solar energy gained) would be as three time bigger. It confirms that the use of electricity for thermal purposes is very energetically unfavourable. Anyhow, the convenience of employing a solar plant is always very big. The energy payback time is less than two years also for a low efficiency plant (flat collector). Considering an expected system lifetime of 15 years the overall solar energy gained is also more than ten times bigger than the energy expenditure for the renewable plant production.

## 7 EMISSION PAYBACK TIME CALCULATION

The emissions payback time is calculated employing the (4). It was necessary to obtain first the values of "$EM_{USE}$" and "$EM_{Conventional}$". These terms were calculated as product of the previous "$E_{USE}$" and "$E_{conventional}$" by the correspondent emission factors ("$ef_{Use}$" and "$ef_{Conventional}$"), see Table 3. It could be possible to calculate a payback time for all the pollutants but it was made only for the Equivalent Emissions that already enclose the main pollutants. The emission Payback Times are shown in table 4.

**Table 3. Emission factors :"$ef_{Use}$" and "$ef_{Conventional}$".**

| | $ef_{USE}$ [kg/GJ] | | | $ef_{Conventional}$ [kg/GJ] | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Gas heater | | | Electrical heater | | |
| | $CO_{2\text{-equiv.}}$ | $SO_{2\text{-equiv.}}$ | O.P.P. | $CO_{2\text{-equiv.}}$ | $SO_{2\text{-equiv.}}$ | O.P.P. | $CO_{2\text{-equiv.}}$ | $SO_{2\text{-equiv.}}$ | O.P.P. |
| Emission Factors | 219,46 | 1,685 | 0,86 | 87,2 | 0,052 | 0,12 | 201,1 | 0,25 | 0,24 |

**Table 4. Emission Payback time for a Thermal Solar Plant in a mono-familiar house.**

| | Emission Payback Time [month] | | | | | |
|---|---|---|---|---|---|---|
| | Gas heater | | | Electrical heater | | |
| | $CO_{2\text{-equiv.}}$ | $SO_{2\text{-equiv.}}$ | O.P.P. | $CO_{2\text{-equiv.}}$ | $SO_{2\text{-equiv.}}$ | O.P.P. |
| Flat collector | 24,1 | <0 | <0 | 1,8 | 11,4 | 1,2 |
| Vacuum 1 | 17,8 | <0 | <0 | 1,6 | 8,6 | 1,3 |
| Vacuum 2 | 18,2 | <0 | <0 | 1,6 | 8,4 | 1,3 |

It is important to underline that in some scenarios the emission payback time is negative: this happens when the yearly emission related to the use of auxiliary energy are bigger then the yearly emission save; the energy payback time for $SO_{2\text{-eq.}}$ and O.P.P. are always negative when a gas-heater is employed; the combustion of natural gas releases small quantities of $SO_2$ and O.P.P. and so the releases saved employing a conventional system are very small; the low energetic advantage of an electrical water heater is reason of a very low energy payback time (sometimes less of 2 months); the emission payback time are generally also smaller than the energy payback times: it confirms the environmental benefit of this RS; as for the energy payback time, the greater efficiency of the vacuum pipe implies lower payback time.

## 8 UNCERTAINTY ANALYSIS AND DOMINANCE ANALYSIS

There are many sources of uncertainty when trying to assess the CED of a material. These influence the final system's CED and it is important to evaluate the differences induced on the final Eco-profile. The CED of many materials was enclosed within a range of values. It is possible that some parameter variations could deeply influence the final result. These parameters could represent the starting point for a following and more detailed analysis. One of the important motivations for developing the uncertainty analysis in energy system scenarios is to capture the different directions of possible future technological change as a result of many technology replacements and incremental improvements. It is also an important information to optimise the Eco-performance of materials ("Eco-design"). The system's CED was so calculated in the different scenarios considering, for each material, the pessimistic and optimistic of its range. The results of this "Uncertainty Analysis" are showed in Table 5, and they are related to three scenarios (optimistic, average and pessimistic). The results of the average scenario are the same already shown in the previous paragraphs.

**Table 5. Results of the Uncertainty analysis**

| | Flat Collector | Vacuum 1 | Vacuum 2 |
|---|---|---|---|
| | [GJ] | | |
| Optimistic | 14,6 | 14,4 | 13,3 |
| Average | 15,8 | 16,3 | 14,5 |
| Pessimistic | 17,0 | 18,2 | 15,8 |
| ***Uncertainty Range*** | ***2,36*** | ***3,81*** | ***2,48*** |

The analysis was conducted employing each time one parameter with its maximum / minimum value and all the others with their averaged one.

**Table 6. Uncertainty Analysis considering the variation of one material per time range.**

| | Maximum Analysis | | | Minimum Analysis | | |
|---|---|---|---|---|---|---|
| | Flat Collector | Vacuum 1 | Vacuum 2 | Flat Collector | Vacuum 1 | Vacuum 2 |
| | [GJ] | | | [GJ] | | |
| PUR | 15,82 | 16,30 | 14,54 | 15,79 | 16,27 | 14,51 |
| Glass-reinf.. | 15,84 | 16,28 | 14,52 | 15,76 | 16,28 | 14,52 |
| Copper-pipe | 15,96 | 16,45 | 14,68 | 15,64 | 16,12 | 14,36 |
| Copper-slab | 16,01 | 16,38 | 14,63 | 15,59 | 16,19 | 14,41 |
| Brass | 15,80 | 16,29 | 14,52 | 15,80 | 16,28 | 14,52 |
| Aluminium | 15,81 | 17,00 | 14,53 | 15,80 | 15,57 | 14,52 |
| Steel | 16,32 | 16,86 | 15,04 | 15,29 | 15,71 | 14,00 |
| Stainless S. | 15,80 | 16,35 | 14,65 | 15,80 | 16,22 | 14,40 |
| Glass | 15,85 | 16,28 | 14,52 | 15,76 | 16,28 | 14,52 |
| Glass (pipe) | 15,80 | 16,38 | 14,64 | 15,80 | 16,19 | 14,40 |
| Stone-wool | 15,81 | 16,28 | 14,52 | 15,79 | 16,28 | 14,52 |
| Propylen-glyc. | 15,98 | 16,46 | 14,70 | 15,63 | 16,11 | 14,35 |

(13) $$Material\,'s\,incidence[\%] = 100\,\frac{\left(CED_i - CED_{average}\right)}{\left(CED_{pessimistic} - CED_{average}\right)}$$

where:

$CED_i$ = System CED obtained by supposing one material with its maximum specific CED value (as in table 6) [GJ];
CED = Average system CED value [GJ];
$CED_{maximum}$ = Maximum system CED value [GJ].

This ratio represents the percentage incidence of a material on the uncertainty range. A comparison between all these percentages allows to obtain the researched "dominant" factor and this type of analysis is called Dominance Analysis (DA). In table 7 were showed the percentage incidence of each material.

**Table 7. Dominance Analysis for each collector's component**

| | Flat Collector | Vacuum 1 | Vacuum2 |
|---|---|---|---|
| | **Material's Incidence [%]** | | |
| PUR | 1,34 | 0,76 | 1,23 |
| Glass-reinf.. | 3,26 | - | - |
| Copper-pipe | 13,62 | 8,44 | 12,98 |
| Copper-slab | 17,87 | 5,18 | 9,15 |
| Brass | 0,25 | 0,15 | 0,23 |
| Aluminium | 0,50 | 37,64 | 0,48 |
| Steel | 43,56 | 30,12 | 41,85 |
| Stainless S. | 0,20 | 3,64 | 9,99 |
| Glass | 3,74 | - | - |
| Glass (pipe) | - | 4,84 | 9,88 |
| Stone-wool | 0,75 | - | - |
| Propylen-glyc. | 14,91 | 9,23 | 14,21 |

Steel is the material with the biggest influence. The big incidence of this material is caused by the great employed mass (mainly tank and pipes) and the big range of the specific CED-values. A further study could be obtained substituting steel with other materials (i.e. copper). Aluminium is another relevant material. The cause can be related to the big range of CED-values that aluminium could have. This material, when is pure, is a very "energy expensive" material. The situation change deeply when is considered a certain recycling rate for which the energy consumption is about one order of magnitude smaller. To decrease the level of uncertainty related to aluminium would be necessary a more detailed investigation about the recycling rate employed for solar collector's production. The last relevant material is propylene glycol. It incidence in the overall result could be related to the not good data quality. This causes an high level of uncertainty and a large variation' range. All the other materials, in consequence of their little employed mass, have a little influence. Glass represents an exception: although it was largely employed in all systems (sometimes more than 35% in weight) the variations of its specific CED-value is not too dominant.

## 9 ENERGY PAYBACK TIME VARIATIONS

The next step was to analyse how the CED variations could influence the payback time. In the table 8 is summarised the energy payback time considering the optimistic and pessimistic scenario. The calculation were made referring to a system with a gas-heater. The percentage reported in the table was obtained as:

(14)
$$Variation[\%] = 100\frac{\left(Payback_{Pessimistic} - Payback_{Optimistic}\right)}{Payback_{Average}}$$

**Table 8. Uncertainty analysis of single-familiar plant.**

| | Energy Payback time [month] | | | | |
|---|---|---|---|---|---|
| | Optimistic | Average | Pessimistic | Uncertainty range [month] | Uncertainty range **[%]** |
| Flat Collector | 21.0 | 22.7 | 24.4 | 3.4 | **14.9** |
| Vacuum 1 | 16.4 | 18.7 | 20.8 | 4.3 | **23.4** |
| Vacuum 2 | 15.5 | 16.9 | 18.4 | 2.9 | **17.1** |

The greater variation interval belongs to the "Vacuum 1" collector: this uncertainty is caused by the uncertainty in the cumulated energy of the employed materials, and in particular of aluminium; It is important to underline that also in the worst scenario the energy payback time is lower than two years. The dominance analysis permit to state the effect on the payback changing the specific CED of a material one at a time. For the studied materials was considered a pessimistic and an optimistic scenario: the former represents the energy payback time considering one material with the higher CED value and all the other on their averaged value; the latter represents the payback obtained with the studied material at the lower CED value and all the other with their averaged value. The percentage difference was obtained as:

(15) $$Variation[\%] = 100\frac{(Pessimistic - Optimistic)}{Pessimistic}$$

The studied materials are: steel in the three scenarios (plant with Flat Collector, Vacuum 1 and Vacuum 2 collectors); copper in the scenario with Flat Collectors; aluminium in plant with Vacuum 1 collectors. The results are summarised in Table 9.

**Table 9. Dominance Analysis and Energy Payback time.**

| | Dominance Analysis [month] | | Difference | Variation |
|---|---|---|---|---|
| | Pessimistic | Optimistic | [month] | [%] |
| **STEEL** (Flat Collector) | 23,4 | 21,9 | 1,5 | 6,4 |
| **STEEL** (Vacuum 1) | 19,2 | 17,9 | 1,3 | 6,8 |
| **STEEL** (Vacuum 2) | 17,5 | 16,3 | 1,3 | 7,2 |
| **Copper** (Flat Collector) | 23,2 | 22,1 | 1,1 | 4,5 |
| **Aluminium** (Vacuum 1) | 19,4 | 17,8 | 1,6 | 8,4 |

It possible to conclude that: if the steel CED was reduced from its high value (pure metal) to its lower value (obtained with a recycling rate of 20%) the energy payback time of the single-familiar plant (with flat and vacuum-pipe collectors) would be decreased from 6 to 7%; if the Copper CED was reduced from its high value (pure copper) to its lower value (obtained with a recycling rate of 40%) the energy payback time of the mono-familiar plant (with flat collectors) would be decreased of about 5%; if the Aluminium CED was reduced from its high value (pure metal) to its lower value (obtained with a recycling rate of 40%) the energy payback time of the single-familiar plant (with vacuum pipe collectors) would be decreased of about 8%.

## 10 SENSITIVITY ANALYSIS AND ENERGY PAYBACK TIME VARIATIONS

The last part of the study was focused on these parameter that could modify the system's $E_{PT}$. The same considerations can be done for the system's $E_{M-PT}$.

### 10.1 Collector Inclination

The previous calculations were all made considering a collector's inclination equal to the place latitude (Italy 38°) as suggested by the collectors producing company. It was so necessary to study the payback time variation obtained by changing the collector inclination of + or – 10°. The results are summarised on Table 10.

**Table 10. Variations of the Energy Payback Time (single-familiar plant)**

| | Energy Payback Time [month] | | |
|---|---|---|---|
| | 28° | 38° | 48° |
| Flat Collector | 23,0 | 22,7 | 23,1 |
| Vacuum 1 | 18,9 | 18,7 | 19,0 |
| Vacuum 2 | 17,1 | 17,0 | 17,3 |

We are able to observe that the variation of energy harvest due to the collector inclination are quite small (about 2%). It is possible to state that the collector inclination has a low influence on the energy payback time and to remind that the user demand was supposed constant during the year. If the demand is changeable with the season it could happen that the studied parameter had a greater influence.

### 10.2 User Demand

The second parameter was represented by the distribution of the user demand during the day. The previous calculation were made considering a minimum demand in the afternoon a two equal maximums on the morning and evening ("changeable" scenario). Three new scenarios was considered: one with a constant demand during the day ("constant" scenario) and two scenarios with the demand strongly concentrated on the morning or late in the evening ("morning" and "evening" scenario). As example, the results for the "Flat" and the "Vacuum 1" collector are shown in Table 11 (the percentage variation is referred to the changeable scenario).

**Table 11. Energy payback time for different user demands of warm water.**

| | Energy Payback Time [month] | | | |
|---|---|---|---|---|
| | Flat Collector | % | Vacuum 1 | % |
| Morning | 24,9 | **10,0** | 19,9 | **6,1** |
| Changeable | 22,7 | **0** | 18,7 | **0** |
| constant | 21,9 | **3,2** | 18,5 | **1,4** |
| Evening | 21,1 | **7,1** | 17,8 | **4,7** |

It is possible to observe that the energy payback time is lower when the demand is concentrated on the evening: this is caused by the higher tank water temperature in the late afternoon; For the opposite, the payback time is higher when the demand is concentrated in the morning. The two symmetric distributions of water demand (constant and changeable) produce similar results; the variation are bigger for the flat collector plant: it is caused by a smaller energy harvest (lower efficiency) and consequently a lower "system's resilience". It is important to remind that "load factor" (300 l/day at 45°C temperature) was considered constant during the year. The differences could increase if the demand is changed. However, in the winter the warm demand increases but decreases the overall water demand; in the summer there is a bigger demand of water but with a lower temperature. It is so not too wrong to consider the load factor constant during the year.

### 10.3 Tank's Volume

There is a little modification in a payback time for volume equal or a little larger than the daily warm water demand (300, 400 and 500 litres). A sensible variation (about 5%) was observed for little volume (200 l): in this case the tank was not able to cover the user demand. The optimum was obtained for a 400 l volume. The results are summarised in Table 12.

**Table 12. Payback time for different water tank volumes.**

| | Energy Payback Time [month] | | | |
|---|---|---|---|---|
| | 200 l | 300 l | 400 l | 500 l |
| Flat Collector | 23,8 | 22,7 | 22,7 | 23,2 |
| Vacuum 1 | 19,7 | 18,9 | 18,7 | 19,2 |

### 10.4 Water Flow

The overall energy harvest have little variations for a large range of water flow values (from 0.8 to 2.4 l/min). The number of working hours was considered the same but the auxiliary energy expenditure was increased for each step of 15% . The results are summarised in Table 13.

**Table 13. Energy payback time for different water flows.**

| | Energy Payback Time [month] | | | | |
|---|---|---|---|---|---|
| | 0,8 [l/min] | 1,2 [l/min] | 1,6 [l/min] | 2 [l/min] | 2,4 [l/min] |
| Flat Collector | 23,1 | 22,7 | 23,3 | 24,4 | 26,1 |
| Vacuum 1 | 19,2 | 18,7 | 18,9 | 19,4 | 20,1 |

### 10.5 Insulation Influence

The insulation's thickness was modified from 1 to 0,5 cm. In this way the overall CED decreased of 0.91 GJ but were increased the heat loss in the tank and in the pipes. The results are summarised in the following table.

**Table 14. Insulation thickness variation and energy payback time**

| | PUR = 1,0 cm | | | PUR = 0,5 cm | | | **Variation [%]** | | |
|---|---|---|---|---|---|---|---|---|---|
| | Flat Collector | Vacuum 1 | Vacuum 2 | Flat Collector | Vacuum 1 | Vacuum 2 | Flat Collector | Vacuum 1 | Vacuum 2 |
| CED [GJ] | 15,80 | 16,28 | 14,52 | 14,89 | 15,37 | 13,61 | -5,8 | -5,6 | -6,3 |
| Energy H. [GJ] | 9,94 | 11,97 | 11,76 | 8,03 | 8,54 | 7,94 | -19,2 | -28,7 | -32,5 |
| Payback T. [month] | 22,7 | 18,7 | 16,9 | 28,2 | 26,8 | 26,1 | 24,3 | 43,3 | 54,5 |

The payback time variations are very big, especially for the vacuum pipe collectors. In fact these collector have a greater efficiency and the flowing fluid reaches an higher temperature: decreasing the insulation thickness, the heat losses are larger than in the flat-collector plant. There is so no energetic convenience on reducing the insulation thickness.

## 11 COLLECTOR SURFACE

The $E_{PT}$ calculation was repeated considering a different number of employed surfaces and the number of working hours was not modified. It is possible to observe that the payback time for small employed surfaces is very high: it is caused by a great incidence of the other system components (supposed constant) on the CED; the diminution of the CED is so lower than the diminution of the saved energy, causing so an higher payback time; the convenience on employing the vacuum pipe collectors is very low for high surfaces (more than 9 $m^2$); this is caused by the low increment of the EH and the constant increment of the CED; the lower efficiency of the flat collectors is shown by the higher value of the energy payback time; on the other hand the lower CED is shown by the lower slope of the interpolating curve; the payback time of the Vacuum 2 collectors are always lower than the Vacuum 1 collectors: this means that the higher collector's efficiency balance the higher CED; the optimal surface value is decreasing with the efficiency of the collectors; it is about 6 $m^2$ for the flat collector, 5,5 $m^2$ for the "Vacuum 1" and 4,5 $m^2$ for the "Vacuum 2". The results are summarised in the figure 5.

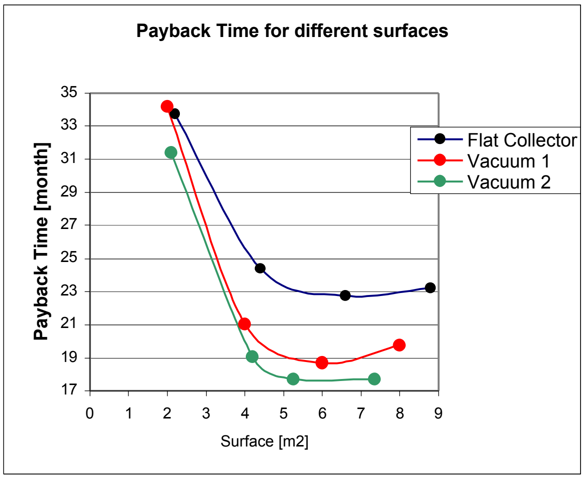


**Figure 5. Payback time for different collector's surfaces**

## 12 CONCLUSION

In order to progress toward sustainability it is unavoidable to measure and compare the resource intensity of processes, products and services. It is even true for the RS, considering that there is an hidden energetic and environmental cost inside these devices. In the paper, the ecoprofiles of some water heating solar devices are showed. CED for single–familiar house is enclosed between 14-16 GJ for different collectors. The Ept are embedded between 16.9 – 22.7 months in the case of gas heater as a support and 4.9-6.4 months for electrical heater as a support. Employing the UA and SA we were able to provide answers to specific questions concerning the behaviour of the system under different limits and assumptions. This is an important information to optimise the eco-performances of solar thermal collector ("eco-design").

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# Approaching The Whole Quality Of Buildings: Methods For The Evaluation Of Economic, Energetic And Environmental Issues

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**Summary: In this work the energy, economy and environment ("three E") requirements for buildings are approached in a joint view, by means of an application to an example building typical of the Italian building stock.**

**With this aim, the rate of involvement of renewable sources is here considered for linking energy and environmental characteristics. Economic and environmental features are joint by means of the evaluation of the cost of the saved $CO_2$. Economic and energy characteristics are linked by means of the cost of the saved energy. In turn, the environmental ambit is addressed by means of the ecoprofile methodology and of the environmental labelling of buildings.**

**This should lead toward a first approach for singling out an overall indicator of the environmental sustainability that, suitably related to other performances of the buildings (e.g., the durability of materials and the indoor performances), could be proposed as a tentative measure of the whole quality of buildings**

**Keywords: Building quality, Energy, Economy, Environment, LCA.**

## 1 INTRODUCTION

It has been several times pointed out that the building and real estate sector has a serious impact on the environment. The construction and the operation of building, in fact, accounts for up to 40% of the energy and materials resources and contributes 40% of the $CO_2$ released in the developed countries. Moreover, a lot of substances dangerous to health come from materials involved in the construction, maintenance and cleaning up of buildings.

The increasing consciousness of such pressure exerted by buildings on the environment opens another demanding job for designers and technicians, along with the traditional tasks concerning the energetic and economic issues.

As it is well known, several methods of analysis are currently available for each of these main fields. Referring to the energy characteristics, for example, the overall heat transmittance of the envelope is one of the most important parameter to be taken into account; at present, it is subject to international and domestic standards. With respects to the economic features, the pay-back time of the design options is probably the most common method adopted world-wide. Finally, the environmental performances of building are currently evaluated by means of the pollutant emissions released by the HVAC system.

But in the field of assessing the overall quality of buildings another question is now arising: how to deal with these important fields of analysis in a joint view? By selecting the sustainability of the buildings as the main goal to be pursued by designers, we will refer here to some environmental features for linking economic and energetic performances.

With this aim, the rate of involvement of renewable sources will be here assumed as the linking parameter between energy and environmental characteristics. In turn, the economic and environmental features will be joint by means of the evaluation of the cost of the saved $CO_2$. Moreover, the environmental issue will be further addressed by means of new methods, like the ecoprofile, that allows the assessment of the whole impact exerted by the building through its life cycle.

In this work the "three E" (Energy, Economy and Environment) issues will be approached in a joint view, by applying some of the previously cited methods to a typical dwelling of the Italian building stock.

This should lead to the singling out of an overall indicator of the environmental sustainability. Such indicator, suitably related to other performances of the buildings, like the durability of materials and the indoor performances (thermal, acoustic, visual and IAQ), could be proposed as a tentative measure of the whole quality of buildings.

# 2 ENERGY, ECONOMIC AND ENVIRONMENTAL REQUIREMENTS OF BUILDINGS IN THE TRADITIONAL APPROACH

## 2.1 Energy issues

Among the various parameters utilised for defining the energy performances of a building, the heat overall transmittance of the envelope can be assumed as the most representative. It, actually, takes synthetically into account the whole thermal features of the building and can be easily utilised to evaluate the energy demand and the energy requirement for the climatisation of buildings through a given period of time (e.g., a season or a month). Not by chance, in fact, several international standards assessed for the evaluation of the energy consumption in the climatisation of buildings expressly refer to this parameter like, for example a recent Italian law (UNI, 1993).

## 2.2 Economic issues

The economic compatibility of the design options is always one of the primary concern of designers and it does not require to be further pointed out. We simply would like to signal here that, along with the costs supported for implementing the design choices, the payback time of these technical solutions should be suitably taken into account.

## 2.3 Environmental issues

Environmental sustainability is today a major issue to be accomplished when approaching technical and policy problems related to the land management. Urban administrations, at this regard, are called to match new requirements, since cities are directly involved in the world attempt of minimising greenhouse gas emissions, as established, for example, by the Kyoto protocol (UNFCC, 1988). In this sense, the building sector represents one of the main fields of intervention.

Therefore, sustainability policies are often embodied in the design process of buildings, by paying more attention to the environmental suitability of the utilised materials and to the pollutant emissions of HVAC system.

# 3 ENERGY, ECONOMIC AND ENVIRONMENTAL FEATURES: A TENTATIVE APPROACH OF A JOINT VIEW

The design and the production of energy-efficient, environmentally sound and economically affordable residences is beginning to be considered an important goal world wide. The Building America Program, for example, through the Consortium for Advanced Residential Buildings (CARB), is currently developing a new innovative home on a community scale (Griffiths and Zoeller, 2001).

Many other methods and interventions could be cited in the filed of the development of cost-effective, energy-efficient residences. Actually, what is lacking is a systematic link among energy, economic and environmental features of a building and the availability of methods able to provide, even at a rough level, a comprehensive judgement of its whole quality. Clearly, this is far to be considered an easy job: but new international standards, particularly concerned with environmental problems, just call for such methods.

In order of pursuing the whole quality of buildings, suitable links among the previous issues should be assessed. By recognising a central importance to the environmental requirements we will suggest here some simple parameters that could be adopted, as a first attempt, for linking the energy and the economic aspects to the environmental ones. This approach could be considered as a tentative address for a  joint view of the “3E” problem in the building design.

## 3.1 Linking energy and environmental performances: ratio between renewable and non renewable energy

With the aim of taking simultaneously into account energy and environmental performances of buildings we propose here a synthetic indicator simply given by the ratio between the amount of renewable and fossil energy sources needed for accomplishing all the HVAC and DHW requirements of the assigned building.

It could be roughly assumed that the greater is the value of this ratio the lower is the impact on the environment. Clearly this assumption does not take into account the pressure exerted on the environment by the renewable energy systems

## 3.2 Linking environmental and economic performances: cost of the saved $CO_2$

The measures adopted in buildings with the aim of preserving the environment are generally supposed to provide a saving in the greenhouse gases released in the atmosphere. These savings, in turn, would involve a change in the costs for the realisation or the restoration of buildings. By ascribing to the released $CO_2$ the role of representative greenhouse gas, we propose to adopt the cost of the saved $CO_2$ as the indicator for the link between the environmental and economic performances of buildings.

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### 3.3 Linking energy and economic performances: cost of the saved energy

On the other hand, increasing the saving of energy in the climatisation of buildings generally does involve a rising of the costs for the construction or the refurbishment of buildings. This suggests to assume the cost of the saved energy as the indicator for the link between economic and energy performances of buildings.

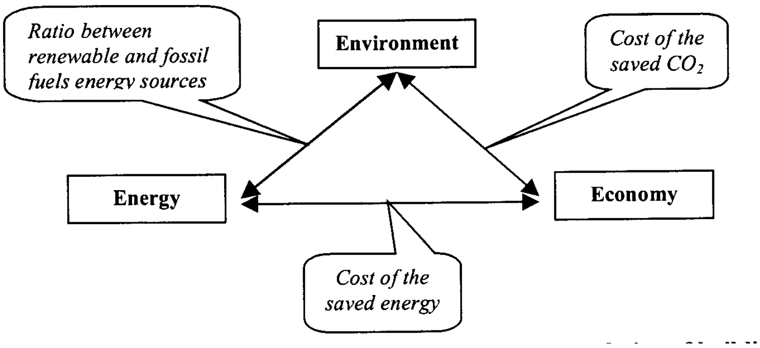



**Figure 1. Visual representation of the "three E" approach in the design of buildings.**

Figure 1 reports a visual representation of the "three E" approach, along with the parameters adopted for linking together each issue, that is environmental, energy and economics.

## 4 FURTHER ENVIRONMENTAL CHARACTERISATIONS: THE LIFE CYCLE IMPACT

Although the amount and the cost of the saved $CO_2$ have been previously indicated as the main parameters for ranking the environmental features of a building, it's obvious that others pollutant matters are emitted during the duty cycle by the climatisation and DHW generation systems; moreover, during the construction phase of the building, several others pollutants are released and a noticeable amount of energy is also involved for the production of the buildings components. In other words, the environmental impact of the building system should be more correctly regarded on the basis of a life cycle analysis. This will enable also the evaluation of the environmental sustainability of buildings.

### 4.1 A method for assessing the environmental sustainability of buildings: the ecoprofile

With the aim of assessing the environmental sustainability of buildings, the methodology of the ecoprofile is gaining a wide popularity among the environmental analysts. It, mainly referring to the Life Cycle Assessment (LCA) procedures, enables the evaluation of energy, materials and air pollution issues involved in the building design, from the building up to the demolition phases (Cole and Ruseeau, 1992; Potting and Blok, 1993; Buchanan and Honey, 1994; Fossdal and Edvardsen, 1995; Trustly and Meil, 1999).

The ecoprofile methodology is essentially focused on the impact exercised by the materials involved in the buildings production and construction on some important environmental precincts. For the building sector the considered environmental ambits of impact are generally the following: the depletion of fossil fuels, the global warming potential, the acidification, the production of photochemical ozone and the eutrofication.

These ambits are characterised through the method by means of suitable indicators. The fossil fuel depletion is usually accounted for by means of the amount of the energy related to the burning of these fuel sources. The global warming potential is generally evaluated by means of the released carbon dioxide. $NO_x$ is supposed to contribute to the eutrophication by means of the chemical oxygen demand (COD). The acidification effect can be assessed by means of the emissions of $SO_2$ and $NO_x$. Finally, hydrocarbons are utilisesed for evaluating the VOCs, that contribute to the photochemical ozone creation.

These indicators of the buildings environmental impact call for the availability of crucial data about the building materials. For example, the evaluation of the GWP requires data of the amount of energy embodied in the materials. All the remaining indicators can be utilised only starting from the knowledge of the pollutant emission factors of materials, that is the emission of a given pollutant released by the functional unit (generally, the mass) of the considered material.

Unfortunately, the lack of data about the embodied energy and about the pollutant emissions released by building materials during the whole life cycle, represents the main constraint for the application of the ecoprofile methodology.

Moreover, available data referring to the embodied energy are affected by a strong unhomogeneity that, in the practice, makes ambiguous the comparison (Cellura et al., 2000) among them and the application to other countries (BRE, 1994; Cole and Rousseau, 1992; Worrel et al., 1994; Boustead and Hancok, 1979).

Emission factors show the same problems presented by the embodied energy. As a consequence literature data (Van de Vate, 1995; Frischknecht et al., 1994; E.U., 1995; Fritsche et al., 1995) should be adopted only for early stage analyses.

In order of overcoming this lack of data, some of the present authors (La Gennusa et al., 2001) have proposed a simple indirect method for assessing emission factors of some relevant pollutants released by the building sector through the whole life cycle. The method starts from the knowledge of type and amount of energy involved in the production of a given material and utilises the pollutant emission factors of each energy source in the assigned country (Onufrio and Gaudioso, 1993).

## 5 TOWARD THE WHOLE QUALITY OF BUILDINGS: AN APPLICATION

### 5.1 Description of the example

The previously introduced approach is in the following illustrated by means of an example application to an apartment belonging to an intermediate floor of a building situated in a mild Mediterranean climate, characterised by 800 degree days (DD).

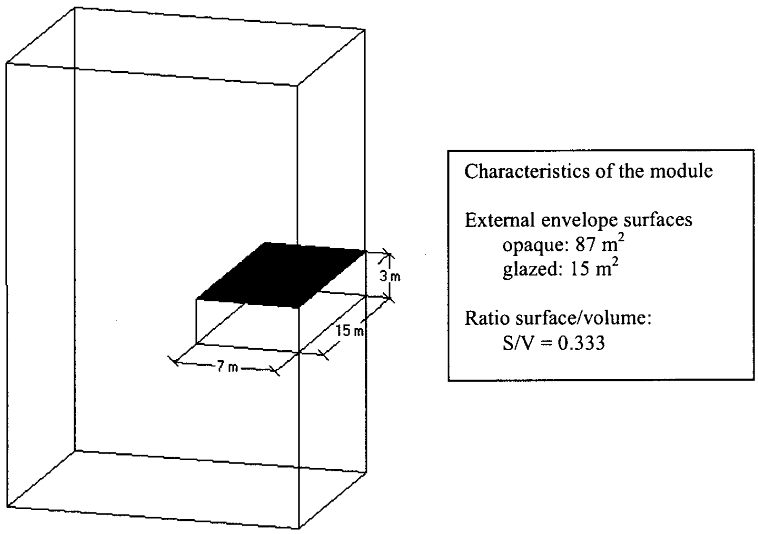



**Figure 2. Geometric descriptions of the example module.**

Figure 2, along with its table, shows the main geometric features of the building module.

In order of singling out the environment, energy and economy features of different design choices, two configurations of walls and HVAC and DHW systems have been selected for the example building. These choices depend on the design characteristics of two cases: case A is characterised by a low building up and installation cost, by a low thermal insulation; case B is characterised by a high initial cost and by a high thermal insulation. Other main assumptions about the cases are summarised in table 1.

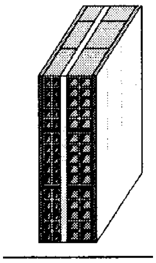
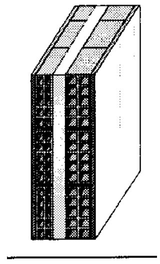
**Table 1. Main design options of cases A and B**

| | Case A | Case B |
|---|---|---|
| Costs for materials production and installation | low | high |
| Thermal insulation of the envelope | low | high |
| Windows characteristics | single glazed | double glazed |
| Summer cooling | heat pump | heat pump |
| Winter heating | oil furnace | natural gas furnace |
| DHW | electric boiler | solar collectors |

Table 2 reports the physical dimensions and the thermal resistance of the layers constituting the vertical walls of the example module.

**Table 2. Physical and thermal characteristics of the envelope walls for cases A and B**

| | **Case A** | **Case B** |
|---|---|---|

| Layer |  | Thickness [cm] | Thermal resistance [$m^2$ K/W] |  | Thickness [cm] | Thermal resistance [$m^2$/K W] |
|---|---|---|---|---|---|---|
| Internal plasterboard | | 2 | 0,03 | | 2 | 0,03 |
| Hollowed bricks | | 12 | 0,27 | | 12 | 0,27 |
| Insulation layer | | 3 | 0,73 | | 6 | 1,46 |
| Hollowed bricks | | 8 | 0,26 | | 8 | 0,26 |
| External plasterboard | | 2 | 0,03 | | 2 | 0,03 |

### 5.2 Environment, energy and economy: the results of the single issue approach

By utilising literature data for the embodied energy and for the emission factor of $CO_2$ of materials and by applying the indirect method previously cited for the evaluation of the emission factors of the other pollutants, we have performed a comparison between case A and case B, for a building life time of fifty years. Data concerning costs of building materials and costs of HVAC and DHW systems here closely refer to the present Italian economic situation (Calvino et al., 2001): this means that absolute values should be considered only as indicative, while results present a good reliability in the comparison between the two cases.

The single issue approach to energy, economy and environmental ambits has been carried out by taking into account separately some different phases of the building management, that is: the realisation of the building envelope ("Building"), the realisation of the HVAC and DHW systems ("HVAC+DHW system"), the aggregate of the previous ambits ("Building+HVAC+DHW"), the duty cycle of the whole building ("Dutycycle") and the whole aggregate ("Total").

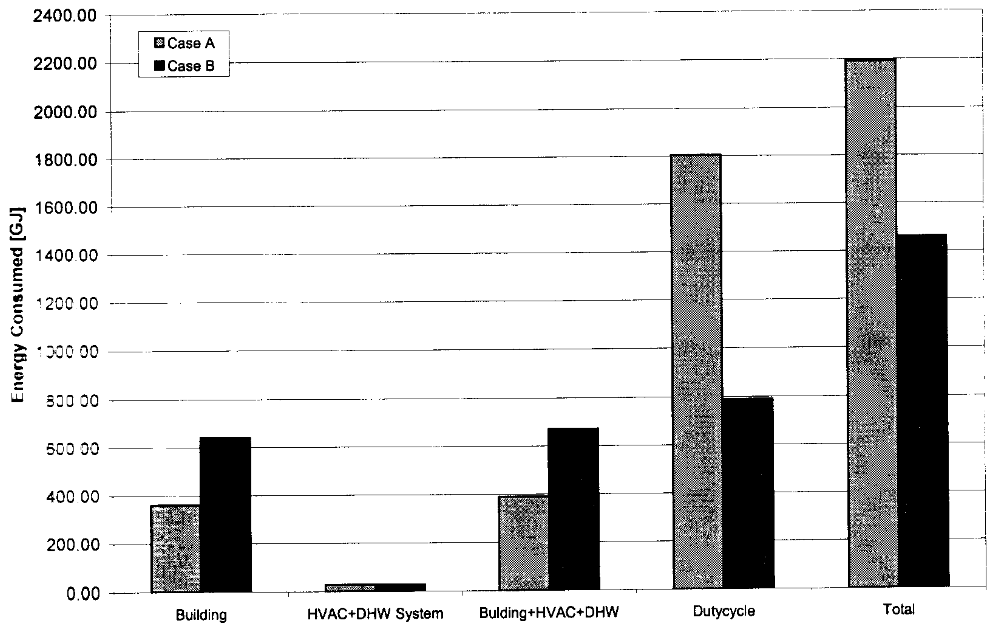



**Figure 3. Energy involved in management of the building modules of cases A and B.**

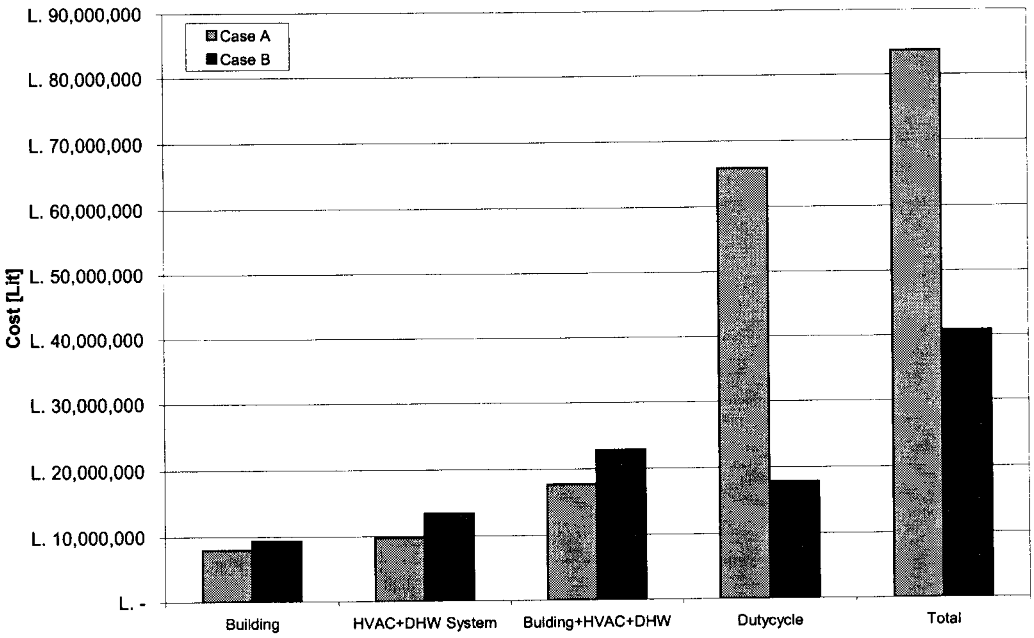



**Figure 4. Costs referring to the building modules of cases A and B**

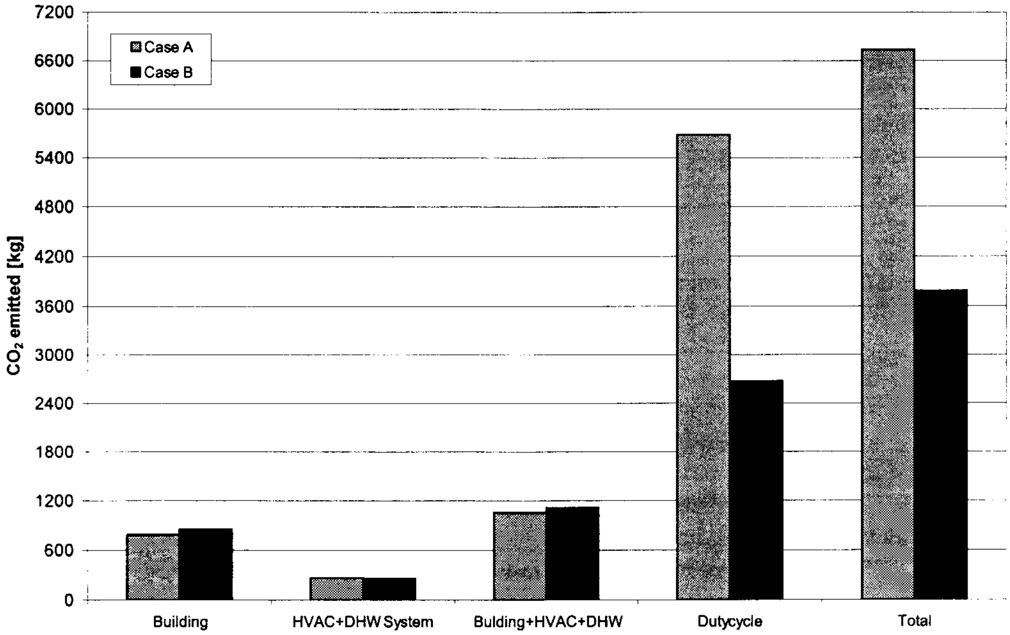



**Figure 5. $CO_2$ emitted by the building modules of cases A and B.**

Figures 3, 4 and 5 respectively apply to the energy, economy (1936,27 Lit = 1 EURO) and environmental issues for the example modules of cases A and B.

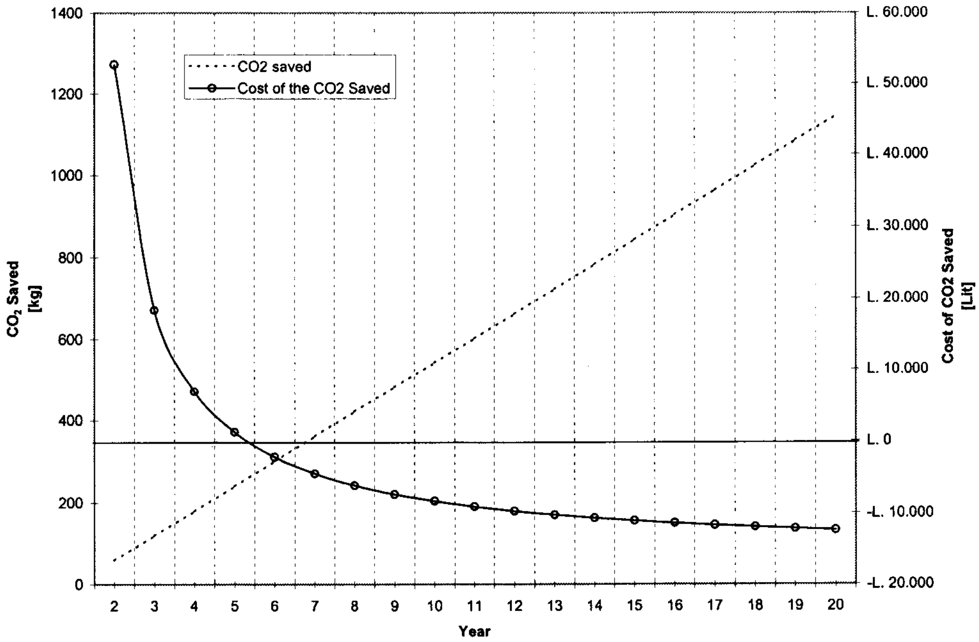



**Figure 6. Payback time of the technical options of Case B.**

Although referring to an indicative example, these graphs suggest some interesting points. The case B presents higher values of energy involved, costs and $CO_2$ emitted in the production and installation phases than the case A; but, by accounting for the duty cycle (fifty years, in this example), it shows a noticeable improvement with respect to case A.

Figure 6 allows the graphical evaluation of the payback time of the design options of the case B, assuming the case A as the reference option.

### 5.3 Environment, Energy and Economy: the results of the integrated approach

The classical approach analysis certainly allows the evaluation of important characteristics of the buildings behaviour (like the payback time and other relevant features), but it is not able to provide a comprehensive view of the whole performance of buildings.

This can be tentatively addressed by means of the integrated approach presented in the previous paragraphs. By applying the method synthesised in Fig. 1 to the building modules of cases A and B, we can obtain the graphical representation of Fig. 7, where on the axes the parameters adopted for linking the energy, economy and environment features are reported: the cost of the saved energy, the cost of the saved $CO_2$ and the ratio between renewable and fossil fuels. In this figure points on the axes represent the values of the parameters for case B with respect to case A.

About the graph, it is interesting to note that the more is the value reached on the axis the more is the impact on the issue represented: in other words, the distance from the origin is a measure of the negative effect of the parameter. For this reason, on the axis representing the involvement of the renewable energy sources we have reported the complement to the unity of the ratio between renewable and fossil fuel sources.

In this way, the area of the triangle constituted by the segments joining the points on the axes can be assumed as a measure of the whole impact exerted by the building on the three ambits with respect to the reference case (the case A, in this example). In this figure the area of the triangle, in incoherent units, is 0.99.

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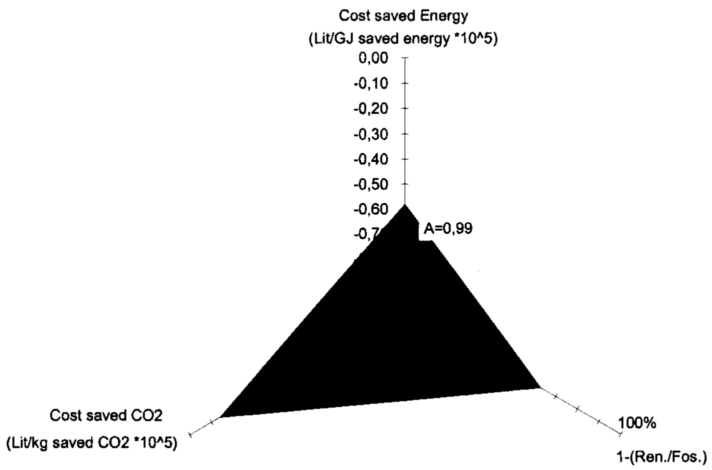


**Figure 7. Graphical representation of the parameters representing the integrated approach of the building analysis (results of case B with respect to case A)**

But, as it has been pointed out, an accurate evaluation of the environmental impact should account for not only the release of carbon dioxide. By means of the indirect methodology of analysis previously cited (La Gennusa et al., 2001), it is possible to assess the pollutant emissions involved in the construction and the management of a building. Table 3 reports the absolute values and the differences referring to the emissions of $SO_x$, $NO_x$, VOC and $CO_2$ for the cases A and B of our example.

Figure 7 and Table 3 can be considered as the main tools for a tentative approach to the whole quality of buildings and for coping with the requirements of the environmental sustainability of buildings.

**Table 3. Pollutant emissions during the lifecycle of cases A and B**

| | | Pollutant emissions | | | |
|---|---|---|---|---|---|
| | | SOx kg SOx | NOx kg NOx | VOC kg VOC | $CO_2$ kg CO2 |
| CASE A | Building - Construction phase | 67,8 | 29,7 | 0,32 | 786 |
| | HVAC - Construction phase + Dutycycle [a] - (gasoline) | 540,0 | 290,1 | 2,97 | 12636 |
| CASE B | Building - Construction phase | 72,8 | 32,2 | 0,35 | 853 |
| | HVAC - Construction phase + Dutycycle [a] - (natural gas) | 57,2 | 68,2 | 4,59 | 2950 |
| DIFF. B-A | Building - Construction phase | 5,1 | 2,5 | 0,03 | 67 |
| | HVAC - Construction phase + Dutycycle [a] | -482,8 | -222,0 | 1,62 | -9686 |
| | Total | -477,7 | -219,4 | 1,65 | -9619 |

(a) The duty cycle here includes the energy amounts required for space heating and cooling and for DHW.

The duty cycle here includes the energy amounts required for space heating and cooling and for DHW.

Of course, graphs like that of Fig. 7 can be usefully adopted for comparing among them several design options. For example, in the case of the presence of two alternatives (in addition to the base case), the values of the areas of the pertinent triangles could be adopted for judging the building whole impact: in Fig. 8 the bigger triangle would show a whole impact of 0.99, while the smaller one would have a whole impact of 0.33, that is remarkably lower.

Clearly the use of the triangle areas for ranking different options is only a rough method for analysing the whole quality of a building: it, in fact, implicitly assumes that the effects of the selected ambits (energy, economy and environment) are characterised by a linear behaviour and that same increments (or decrements) on different axes have the same impact on the ambits. This is really hard to be stated. At this stage, the triangle characteristic of a design option must be adopted only as a visual synthetic representation of the whole impact of a building, while the parameters reported on the three axes represent an actual measure of the impact on the issues that they are supposed to link.

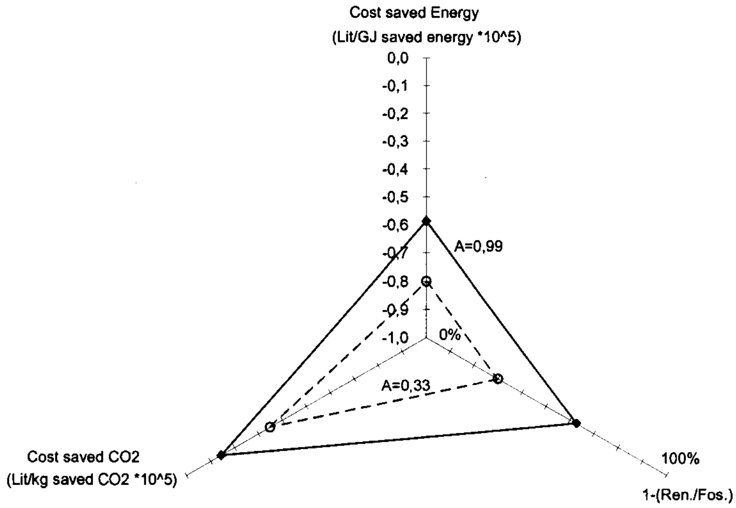


**Figure 8. Triangles representing the whole impact of two different design options with respect to a base case**

## 6 CONCLUSIONS

But another question now arises: can be considered the environmental, economic and energetic analyses here depicted as the only judgement elements for ranking the suitability of a building design, or should them be coupled with other issues referring to the indoor performances realised by a building?

Actually, the main goal of a building is to provide a safe and comfortable space where people can perform their work and life activities. As that, the indoor performances of a building should be usefully embodied in the analysis concerning its whole quality: acoustical, thermal, visual and air quality issues should be taken into account, along with the previously treated parameters. This would lead to synthetic representations like this reported in Fig. 9 (Nucara et al., 2000), where the main aspects of the indoor quality are reported.

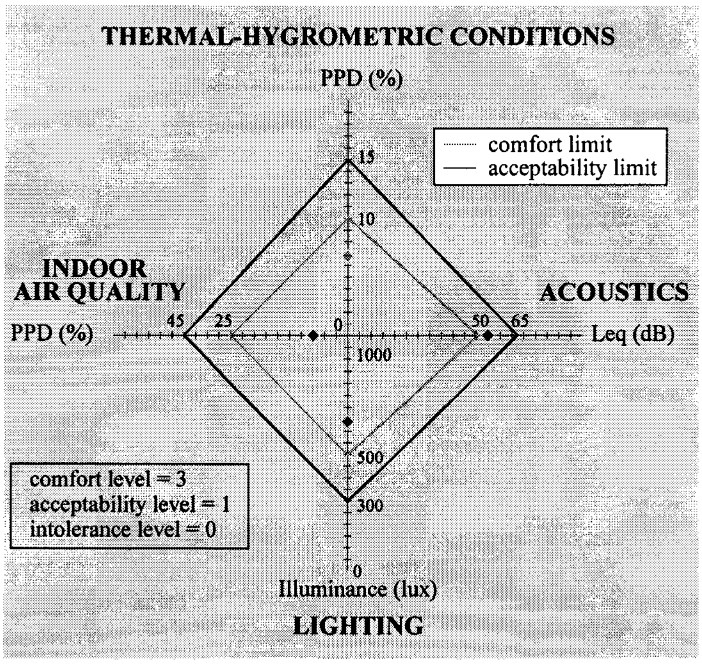


**Figure 9. Example of *quality diagram* referred to one of the schools examined.**

A suitable link of graphs like that depicted in Fig. 9 with graphs like that of Fig. 7 would allow to address the problem of singling out a suitable indicator of the whole quality of buildings.

This, in turn, should be further related to other important performances of the buildings (e.g., the durability of materials), and to performances referring to the urban facilities (e.g., the access to the public transportation network).

These simple notes shows that the obtaining of an unique indicator of the whole quality of a building still requires a deep and long research effort, if ever it should be possible to attain it.

But the search for more useful methods and parameters for synthetically ranking the performances of buildings is a fascinating way that surely deserves the efforts of researchers.

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# Disability-Impact Formula For The Health-Supportive Home

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**Summary: The home generally functions as a protective shell against threats. Economic use of the sustainable product "dwelling" implies a continuous, functional performance of applied building materials and constructions within the whole exploitation period of the (healthy) dwelling.**

**Demographic changes in society call for interventions. Building performance is threatened by both the changes of design-criteria during the exploitation-period and by degradation of the building. A prominent presence of dwelling-related- diseases in modern dwellings indicates the effects of construction-errors on health-conditions in the living environment. Health preventive intervention in building constructions declines its effects; high costs for medicines and health-care. Outlined is a causal principle for health-supportive prevention. Design-methods provide affordance-theory for activities in relation to interventions in the physical environment. The impact of interventions on avoidable costs for care service forms a starting-point for purposive durable design. Affordability of public efforts in health-supportive building-technology however lacks an economic equitation formula.**

**Assessed is in this paper an algorithm, relating the economic-impact of a health- supportive intervention in the house stock with savings in public costs for medical treatment and care-assistance.**

**A theoretic Public-Health framework for health-supportive intervention is briefly described.**

**In Building-Construction Risk-Analysis the Torroya-algorithm relates prevention costs with the product of demolition-risk and the sum of direct rebuilding-costs plus indirect public costs. Generalisation for building related disability-risks and substitution for health factors provides an elaboration of a Disability-Impact Formula. The public benefit of an investment in health supportive intervention in buildings is related to the product of disability-risk and disability-impact. For dwelling- related diseases the impact of interventions is estimated to be the difference between actual quality of life level and full health. Weighing in the formula is renounced for precocious mortality. Correction- factors for appreciation and pathogenic relationship between physical environment and disease are introduced in the formula.**

**Equitation of one year PDAYE with the avoided effort for care-services provides a break-even point for investments in the house stock and avoided costs in Public Health. A strict spit in the configuration of the formula results in a multidisciplinary tool. Public health will provide data that result in a sum of potential savings. Investments in the house stock are balanced with this given budget for health-supportive provisions of the dwelling. Application of the formula is restricted by privacy legislation to the scale of a whole population, its house stock and global demographic data for disabilities.**

**The Disability-Impact-Formula will enable further research based on simulation of the effects health-preventive provisions in building-constructions.**

**Keywords. Healthy home, design-intervention, affordability, durable investment, predictive simulation**

## 1 INTRODUCTION

The ageing West-European societies are faced with the effects of declining health and vitality, increasing costs for medical treatment and care-assist for impaired and disabled persons.

In the Netherlands the part of population older than 65 years will grow more than 20 percent of the total population while up to 70 percent of this cohort has one or more chronic diseases. (CBS, 1996). Ongoing ageing of the population threatens the economic basis for investments in housing.

Ageing confronts public housing with massive problems in relation to the existing house stock and the foreseen demographic changes. In the Netherlands the group above 65 year old is indicated to grow from 2 million persons (15 percent) in 1995 to a maximum of 4,5 million (25 percent) in the year 2040 (CBS, 1998). With an average growth of 1 percent of house stock, (WBO, 1993) the complete addition of house stock is needed during the coming decades for the purpose of housing older growing persons. Since public health is a main task for the public administration, inclined investments in building construction introduce a discussion concerning the affordability of health-supportive design-solutions.

A key issue is whether it will be possible to prevent for the huge amount of disabilities by a smart design of the dwelling? What intervention in disability impact is possible with a Healthy-Home (Lotz,1974) design?

### 1.1 Appropriate dwelling design and prevention for disabilities

Avoiding the expression of disabilities is defined as "primary prevention". Tertiary prevention is defined as an avoidance of the public effects of actual expressed disabilities ( RVZ, 1996). Health-supportive protection by building engineering is a matter of primary prevention as well as "tertiary prevention", which is focussed on the avoidance of indirect damage resulting from an already expressed disability.

> I.e. provision of a dwelling plan suitable for use by wheelchair-users prevents these users for further expression of effects like **1.**social isolation and **2.**dependence of care-services in Activities-of-Daily-Life (ADL). In our wheelchair-example, the objective is to prevent persons to be forced to use a wheelchair. The possible means are found in avoidance and elimination of the causes of low mobility and vitality: obstacles, thresholds and causes for slipping and tripping falls in the building.

Building engineering focuses on primary prevention of avoidable disabilities as a result of unhealthy conditions in the building. An appropriate building-performance is threatened by both a gradual under-performance of the product "building", compared with its initial design-concept and by the changes of design-criteria during the exploitation-period. A Life Cycle Analysis of a building has to take in account both the actual requirements and the future changes in use during its planned exploitation. The effects of changing demographics and declining health-status on the house-stock however can be foreseen. The Healthy home theme is a traditional tool in sustainable architecture that may provide solutions. Popular stated, as an architect is not a physician, the scientific basis for a healthy-home is weak. This paper regards the healthy home to be a health-supportive tool.

### 1.2 The Healthy dwelling theme and sustainability

According to the Triple P-principle (Atkinson 1976) a sustainable world is to provide sane conditions for [intrinsic] Profit [including all hidden costs] [the material resources of] Planet and Persons on the long term. In the context of this paper we focus on the Persons aspect.

The home implicit functions (Rapoport, 1975) as a health-supportive enclosure that shelters the indoor against all kinds of outside dangers, storm, rain too much sun, too low temperatures, predatory beasts, parasitic bugs and quarrelling humans. Thus it enables the human being to develop a stable setting for optimal social functioning. (Van Bronswijk, 2000). Optimal social functioning has also grown a goal in medical treatment. WHO (1946) defined health being "a state of complete physical, mental and social well-being and not merely the absence of disease and infirmity". The home is to provide conditions that support health and wellbeing by accepting the human dimensions, capabilities and all the physiological, biological and socio psychological limitations of humankind; providing a basis for environmental design (Preiser and Visscher, 1992).

### 1.3 Health and the competence of architects

Expression of a disability according to an disabilities-determinants model VTV 1997 (RIVM 1997) of the "Dutch National Institute for Public Health and Environment" depends on the presence of both heredity-aspects in the individual human body, called endogenous disability-determinants, and environmental aspects outside the human body, called exogenous disability determinants. Social environment, lifestyle and physical environment are exogenous disability determinants. The competence of architects for the health-aspects is merely found in a purposive manipulation of the built environment, being a part of the physical environment. Actually, all architecture is intended to be health-supportive intervention. (Heeg, 1994).

### 1.4 The rationale in health-supportive intervention

The physical environment conducts the condition for comfortable healthy human living, but Homo Sapiens shares this appreciation for comfortable growth conditions with several biologic disability-agents like bacteria, viruses, moulds, mites and insects (Van Bronswijk, 1998). Primary prevention against disability- risks concerns elimination of those features in the architectonic configuration that afford a disability-agent to express its symptoms. Purposive prevention proposes the existence of constraints that relate to the level of individual disability- risk and sets of conditions that afford the expression of a disability.

The combination of properties that allow an organism to express a certain activity for good or ill, is defined an Affordance-Pattern [Gibson, 1992]. An Affordance in architecture is proposed by Tweed, [2001] to be a pattern that relates the set of properties of an architectural configuration (or its enclosed indoor-climatic conditions) and the set of expressed activities that is afforded.

Health-supportive intervention thus becomes the simultaneous implementation of two affordance- patterns in the design of the dwelling. One pattern affords comfortable conditions for the human being, while another pattern provides a building-performance that is hostile to human threatening agents.

The data-sources for these patterns however are found in biology and medicine.

> I.e. Long lasting humid and warm climatic conditions provide an Affordance for a growth-explosion of the disease-agent mite; while the same climatic conditions give an Affordance for the human being to express allergic reactions under condition of a growth-explosion of mites. Intervention in the physical conditions, that simultaneously afford continued comfortable use by human beings and not-afford excellent growth of mites may be installed by dry indoor-air conditions i.e. floor- heating (which offers warm, dry conditions). (Koren, 1995)

### 1.5 Evidence for the impact of dwelling related diseases.

For each separate health-threat a multi-disciplinary research should be started towards the interaction between disability-agents and building-performance, interventions should be defined and tested. This is to be done for exotic health-threats as well as for very common health threats with high public impact. Evidence for the impact of individual disabilities is required in order to be able to judge programs for health-supportive intervention in the house-stock. I.e. Nowadays elderly people stay up to 95 percent of time indoor, continuously being exposed to noxious air conditions.

## 2 PROBLEM AND AIM

Health- preventive intervention may decline the effects of disabilities at high costs for medication and care-assist. The scale of required public efforts for care-assist is a major problem the ageing Western societies. A problem occurs, since the prevented costs in the domain medical care are the result of additional prevention-investments in the domain of building engineering. A sustainable approach from point of view of Profit on the long run for a whole society including indirect prevention investments and indirect care-assist costs is needed. Besides not all proposed investments in health-supportive interventions will prove to have an economic profit. The benefits of intervention have to be in balance with the affordability of public investments (Regnier, 1999).

### 2.1 The problem is a lacking balance of health-benefits and building-investments

Affordability proposes an economic equitation based on an evaluation between the investment of prevention-costs in a building-construction and the benefit of an avoided health-risk (premature mortality, medical treatment and care of avoidable dwelling-related disabilities). An investment in prevention-costs assumes the existence of an alternative validated intervention. The prevention by building engineering assumes competence in health-supportive intervention by building engineering. Solving the affordability-problem presupposes the integration of medical sciences, biological sciences and architecture. Such an affordability formula for health-supportive intervention is still missing.

### 2.2 A formula that relates medical impact of disabilities and optional interventions in house-stock

In this paper is being developed an algorithm for the evaluation of the affordability of health-supportive interventions in the house stock; interventions that prevent for the impact of dwelling related disabilities on public health-care efforts.

### 2.3 Research questions

What context positions the affordability of health- support?

What is an effective notation of the public impact of disabilities?

How is a constraint for public affordability of interventions being deducted from existing models?

Whether the deducted formula is an suitable starting-point for architects and planners of house-stock?

## 3 POSITIONING OF PUBLIC AFFORDABILITY OF HEALTH-SUPPORT

Historically health was regarded to be merely the absence of diseases and impairments and the research immediately focussed on elimination of distinct diseases. The absence of disease and infirmity is located in the absence of symptoms, resulting from a variety of disease-causes.

Since health-threats concern both the effects of disabilities, impairments, chronic diseases and accidents, they are brought together in the term "Disability".

Screening of the relation between health-supportive intervention, conditions in the dwelling, building construction and prevention originates from the Public Health Engineering. The health engineering was established founded in Europe as a reaction on unhealthy living-conditions in the first mass housing initiated by the Industrial Revolution. The efforts for systematic achievement and assurance of healthier living-circumstances resulted in Building Codes in the beginning of the 20th Century. In the first versions of the Codes, restrictive patterns for the built construction were defined, extorting circumstances that hardly afforded an unsafe and unhygienic life-style of its inhabitants. The efforts to improve the control over the individual life-style however conflicted with individual goals and interests of users. The public discussion about the affordability of preventive provisions in the built environment was made on the level of social vision, rather than to be calculated in risk factors and their underlying causal relation with the design of the built environment.

The (1946) WHO defined health in two phases; at first, as a state of physical, mental and social well being and at second: not merely the absence of disease and infirmity. Complete health should imply the complete absence of any disability. The EEC - Directive for Building Products and Constructions (EEC, 1998) orders, that the building has to be designed in that way, that the building itself does not contribute in an additional health-risk. Its implication is an elimination of conditions in the building construction that contribute to the health-risk through an appropriate application of building materials and constructions.

### 3.1 The risk for collapsing of a bridge as an example for a health-threat

A historical analogy of a health- supportive standard, based on a risk-assessment in dwellings is found in the Building Code for the mechanical aspects of the building construction. The physical-, mental- and social effects on the human being, resulting from a collapsing built construction are mortality or decreased social functioning as a result of fractures, remaining impairments, mental shock or lost living conditions. In the WHO (WHO, 1946) definition the effects were regarded to be health-effects. In his Philosophy of Building Construction (1953) Torroya argued, that collapse of a public work (such as bridges) causes economical damage. The damage is a sum of direct costs $C_d$ for rebuilding of the lost construction, (the block B in scheme 1) and indirect Costs $C_I$ (Block C in scheme), resulting from the collapse. Indirect public Costs $C_i$ should be taken into account in the investment-proposal.

In Building-Construction Risk-Analysis the Torroya-algorithm [Torroya 1953] related Prevention Costs (Block A in scheme 1). Acceptance of the standards in this formula is a matter of balance between the affordability of a public effort in financing preventive provisions and the risk-level of possible (direct and indirect) effects of a failing building-construction. Prevention-costs for a public construction (Block A) and costs for indirect public effects (Block C) are regarded a simple matter of public accountancy; both expenses have to be financed by the same public administration in the case of a bridge.

The initial Torroya-formula contained 3 main components that were related:

- Block A, involved with investments in additional preventive technical provisions in the bridge;
- Block B, the product of risk for, and rebuilding-costs effect of a collapsing building-construction;
- Block C, the product of risk for- and indirect public costs of a collapsing building-construction

In a formula; prevention for direct and indirect effects of construction failures

*Prevention Costs* *Risk ∗ ( sum of direct costs and indirect costs)*

$$C_p \quad <= \quad R * \quad (C_d + C_q)$$

*Which allows a redefinition :*

$$C_p \quad <= \quad R * C_d \; + \; R * C_q$$

*(Block A)* <= *(Block B ) and (Block C)*

in which is;

| | |
|---|---|
| $C_p$ | Costs for prevention invested in the building construction; |
| R | Risk for a combination of forces and conditions, causing collapse of a building; |
| $C_d$ | Costs, direct made for rebuilding the demolished building; |
| $C_q$ | Costs, indirect made as a result of the collapsed building. |

**Scheme 1. Torroya-algorithm for Preventive investment in building construction**

### 3.2 Established generalisation public task from bridges to building

A first generalisation was made from a public work into private financed investments. The generalisation towards a weighting of individual private building-collapse prevention costs and the indirect public costs was made through the acceptance of the Torroya-formula within the Building Codes for all Buildings. As a direct result of the Torroya-formula the risk-factor analysis was accepted as the basis for standards in Building Codes. [TGB] The responsibility for preventive provisions that avoid indirect public costs (Block A) was embodied in user-oriented legislation. Principals of buildings from now on have to deal with it by providing an appropriate health-supportive building. Application of the formula will represent the practice if a financial claim towards building-initiators is established. The additional private investment in appropriate prevention-provisions in the building-construction is an early application of the principle: “The originator of environmental damage is responsible and is the one who pays the bill ”.

### 3.3 Generalisation of responsibility to all health-threats, related to the building construction

In actual developments in Building Codes and the earlier mentioned EEC -Directive for Building Products and Constructions (EEC, 1998) motives are found for the investments in our house stock of health-supportive provisions. Both the configuration of risk-analysis in the Torroya-formula and its actual established implementation in Building Codes deliver a basis for further generalisation.

The responsibility is only valid for that disabilities, that are causal related with the design of the building itself. For collapsing bridges the relation between health-threat and quality of construction is evident. For general purpose a first correction- factor for Evi**d**ence of **d**ata d is introduced.

## 4 GENERALISATION OF THE TORROYA –FORMULA TO ALL HEALTH-THREATS

The structure of the formula enables a generalisation towards health-supportive preventive investments, which intervene in all kind of health-threats.

The formula has to be written for each individual health threat with the product of demolition-risk Ri and the sum of all Direct rebuilding-Costs plus Indirect public Costs (Block B and block C in scheme 2) during the Life-Cycle of the building.

The formula can be defined as a sum of risks for all individual health-threats:

- Block A, involved with the features in preventive provisions in the building-construction;
- Block B, introducing the risk of loss of the building-investment resulting from a health-threat i;
- Block C, focusing on the risk of indirect public costs resulting from an individual health-threat i.

The correction-factors F compensates for the phenomena that applied provisions not automatically imply a beneficial use. Consensus of opinion about risk-levels, accepted in the society, forms the basis for standards for failing building constructions. The aim is prevention of the risk for effects of health-treats up to a risk-level, where public acceptance of health-effects is in balance with the affordability of individual investments in preventive provisions of the building.

### 4.1 Direct generalisation of collapse of constructions to all health-threats

An issue is, if it is allowed to generalise the risk for mechanical failures of building-constructions to all health-threats? A building-collapse acts different from a bacterial infection or falling from the stairs. In a direct way, health-risks are implemented in the Torroya-formula. The potential of the formula in scheme 2 is in an integrated estimation of the complete health-effect. Direct translation of the formula specified for the case of collapsing bridges resulting from the combined effects of bearing loads on a building and gradually devaluating mechanical features is theoretically possible.

### 4.2 Starting points for generalisation of the health-threat on the level of the building

#### 4.2.1 Block A prevention investments in a whole house stock

Block A focuses on the investment in health-supportive provisions. Elaboration of this part of the formula will regard optional solutions in public housing for design and distribution of preventive interventions. The relevant domain of research is architecture.

The investment in a whole house stock is the product of the number of dwellings in the house stock $H_n$, the part of house stock that is involved in the intervention and the mean costs of an individual intervention.

#### 4.2.2 Block B Direct costs of rebuilding

Block B represents the product of the risk for building-collapse and its resulting rebuilding costs.

A focus on the Quality-of-Life of the inhabitants pleas for neglecting the rebuilding costs in a formula for affordability of a programme for preventive provisions in the built environment. If one disability is unfit, this does not imply that the environment is unsuitable for human use. Rebuilding of the collapsed building is an unavoidable but peculiar cost-effect of analysis based on a continued performance during its planned exploitation period.

A generalisation of rebuilding within the block prevention- costs is possible. Allow an investment, that assures a budget for rebuilding for a collapse with accepted risk R, and a prevention for the remaining risk-period (1-R). The implication is a neglect of the block B rebuilding costs, being one of the optional interventions.

### 4.2.3 Block C Public impact of disabilities and impairments

Block C focuses on the public impact of disabilities and impairments, written down a risk-analysis. Elaboration of this part of the formula depends upon processing of health-data and is within the competence of Public Health sciences. Two kinds of problems occur in the generalisation to all health risks: the availability of medical data and the achievement of a proper health-risk.

### 4.2.4 Availability of medical data in block C

Recent Privacy-legislation declines the number of combined data for smaller groups since individual medical records of individual subjects are destroyed after a period of ten years. An incomplete record complicates the research towards the long-lasting effects of dwelling conditions on individual persons based on their medical records. Direct measurement of the medical effects of an intervention in housing requires a multidisciplinary approach of both Building Engineering and medicine. A research on the body toward vitality- or health-effects however is restricted in a medical protocol to physicians.

### 4.2.5 Achievement of the health-risk in block C

Assessment of the risk-level has to deal with two aspects: determination of the gain by an intervention-programme, and determination of the intervention level at borderline, intrinsically hidden in the actual building-typologies.

A Biologic risk is not a constant factor, but tends to grow, afforded by good conditions.

The performance of building-construction depends on its fit to the local outdoor-conditions.

Since conditions in house-stock vary with both the design and the behaviour of the inhabitants, detailed information of relations between health-effect and design in literature is rare.

Besides, the offer of a preventive provision does not automatically imply using the benefits of it.

Corrections have to be made for evidence of dwelling-related –disability d, for appreciation of provisions; compliance of an intervention-treatment. Earlier studies on acceptance of interventions provide factors for accommodative behaviour, which is associated with an acceptance that varies from 18 %for home maintenance to 88 % acceptance security and communication (de Kort, 1999).

## 4.3 Strategy for application of Disability-Impact Formula

For each individual disability is constructed a balance of investments and public benefits of an intervention. The block B Rebuilding- Costs is regarded a peculiar block, initiated by the original example, collapsing bridges. The investments are to be regarded on the level of a house stock during its whole exploitation period. The assessment of the public benefits as a product of risk-level, individual costs per subject and costs per subject give some practical objections.

The remaining formula exists of three parts:

- Block A: describing a distribution of individual prevention-investments in a house stock;
- Block C: describing the theoretical benefit of maximal intervention which is equal with the Public impact of disabilities;
- Correction factors F, dealing with the difference between theoretical- and practical benefit.

These operations result in a two-sided formula that provides a Tool for a multidisciplinary approach.

The Tool connects a public- housing part and a public-health part:

- For Public housing: variables in Block A for Preventive Investment are smaller than a budget.
- For Public-Health: the budget equals prevented costs in care-assist, beneficial from intervention

For the purpose of positioning all architectural items in the left side, and all items that refer to human studies in the right side, in chapter 5 is focused in detail on the assessment of the impact of individual disabilities, based on existing items in Public Health. In chapter 6 the focus is on the distribution of investments in the house stock.

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## 5 ASSESSMENT OF IMPACT OF DISABILITIES (BLOCK C OF FORMULA)

In Block C for the Public Health-Effect a conversion is possible, in which costs are related to the product of gain of Avoided Loss of Years-of-Life and the costs for one year of provided care-services as a compensation for a disability. The gain in Quality-of- Life is a discriminating factor for the impact of a provision. Actually a potential gain of Quality-of-Life represents a prevented loss of Quality-of-Life. Since chronic disabilities are chronic as a result of unavailability to cure, the effects of chronic disabilities and impairments are centred in the last part of life. Prevented loss of Quality-of-Life avoids a compensation by care-assist, needed to maintain an appropriate Quality-of-Life. The prevented costs for compensation in one reference full year of care-assist are defined to be the average costs of 1-year stay in a Nursing-Hospital. The impact of a disability becomes the product of all reference-years of preventable disability and the costs of one reference year care-assist within a population. The factor reference-year of prevented disability itself is a product of the frequency in the population, the average difference in gravity and the average prevented length of the disability.

### 5.1 Weighing the impact of a disability

The impact of disabilities can be weighted in several ways, starting with a notation of the risk.

The product of population and frequency gives a more detailed view upon the impact of a disability. In Public Health Engineering for individual disabilities, the product of risk and population is given in the term Prevalence (for chronic diseases) and in Incidence (for the accidents and temporary disabilities). The product of the number of impaired persons and their period of impairment provides additional information. Nevertheless, the gravity of impairment is not equal for all persons. A more detailed value for the impact is assessed by the introduction of a weight for gravity of disabilities. Weighting levels of gravity allows measurement of gradual increase, resulting from an intervention.

In Public Health a product of prevalence, mortality and weighted gravity of apart disabilities is introduced in 1993, the Disability-Adjusted Life Years (DALY). The relative public impact of morbidity as a result of a disability can be calculated in Disability-Adjusted-Years. The DALY method results from the WHO Global Burden of Disease study. (Murray, 1996).

Formula for Disability Adjusted Life Years DALY

**DALY= LYLi + DAYi * Wf,i**

| | | |
|---|---|---|
| DALY: | Disability Adjusted Life Years, caused by disability i | in years |
| LYL: | Lost Years of Life for disability i | in years |
| DAY: | Disability Adjusted Years contributed by disability i | in years |
| Wf,I: | Weight Factor for Quality-of-Life , varying from healthy 1 to 0,03 for terminal state | |

**Scheme 2. Factors, contributing in Disability Adjusted Live Years (DALY)– formula**

The DALY formula results from a weighting of both premature mortality, the Lost Years of expected Life (LYL) and the DALY, LYL, DAY and Weight factor Wf (Stouthard, 1997). All these factors concern the impact of individual disabilities on the scale of population. The relations between DALY and its elements LYL, DAY and Weight factor $W_f$ is given in scheme 2.

> I.e. Demographic data in the Netherlands for 54 disabilities in 174 levels of Weight are available for import in the Disability-Impact-Formula. The data indices in a rather accurate way the medical situation at baseline before a preventive programme for health- support is initiated.

Use of the DALY-technology opens within one population the opportunity to compare gravity of individual disabilities and changes in impact in a range of years. The comparison in terms of DALY eases the determination of effectiveness of intervention-programmes.

### 5.2 Weigh factors for dwelling- related disabilities and non-related disabilities

Interventions against disabilities that are causal related to the artificial indoor-climate conditions in the building environment allow for primary-prevention. Intervention by limiting the Affordance- in the human living-environment to a part, that is hostile for growth of disability-agents, theoretically allows a complete avoidance of the disability. The theoretical gain in the disability weight-factor Wf is 1-Wf for disability. Which assumes the possibility of complete recovery.

Non-related disabilities as well as dwelling related diseases have the potential for tertiary prevention. The actual level of quality of life is taken as reference. Protection against further loss is the goal of intervention. Improvement in Weight factor is no goal and in this formula the gain in Weight factor Wf for non-related disabilities is therefore set to zero.

### 5.3 The impact of a disability conversed in a budget for intervention

A budget for an intervention can be defined by the product of five factors: at first Prevalence of a disability, at second the lost Quality-of-Life defined in Weight $W_f$, at third the period of benefit of intervention and at fourth the mean yearly costs of 1 year full medical care-assist in a Nursing Hospital at fifth a set of correction-factors F in Public Health for appreciation and compliance.

The period of theoretical benefit at a maximum equals the period of being a recorded patient, presumed that no intervention is executed. The benefits of an intervention count from the moment the intervention is successfully implemented.

## 6 THE OPTIONAL DISTRIBUTION OF PREVENTIVE PROVISIONS (BLOCK A)

The distribution of a given investment-budget for preventive provisions in a house stock depends on the prevalence of the intervened disability for efficiency reasons. For this study we assume that costs for preventive provisions for an individual disability i are the product of the number of intervened dwellings and the costs Cp,i for an individual provision in one dwelling.

The maximal intervention is conducted by a general application of the intervention in the complete house stock. Efficient implementation of interventions in the building-process is possible, however efficacy of use may vary from 4 to 15 percent of dwellings.

For this purpose, in block A of the formula are introduced the following terms:

- N The number of dwellings of the house-stock;
- $H_n$ The house stock
- $\phi$ The average number of inhabitants in a dwelling;
- g Factor for Implementation Rate of preventive provision in the house stock

The number of dwellings involved in an intervention programme is the product of both the number of all dwellings in the house- stock $H_n$ and the implementation rate g

The total investment is the product of numbers of dwellings in house stock Hn, the implementation rate g and the mean individual costs Cp,i.

## 7 THE RESULTING DISABILITY-IMPACT FORMULA

Equitation between the prevention-investments in Block A and the avoided public-costs in Block C is elaborated in chapter 4. A detailed structuring of public impact of interventions is developed in chapter 5. The distribution of the preventive provisions is elaborated in chapter 6. The resulting formula can be found in scheme 3.

In formula;
during exploitation period:
intervention-costs parameters < the product of prevalence, period, and yearly costs of an disability
Prevention block A < Factors F x Public Health Disability Impact x Annual Costs

| | | | | |
|---|---|---|---|---|
| *PREVENTION* $\underline{H_n * C_{p,I}}$ [m*g] | < | [Factors F]*(Prevalence)*(1-W $_{f,i}$) *DISABILITY - IMPACT* | *($P_i$-$P_b$) *PERIOD* | * $C_{a,c}$ *SAVINGS* |

*in which is;*
$C_{p,i}$ Costs for intervention
$R_i$ Risk for expression Construction-Failure i
$C_{d,i}$ Costs, direct made for the repair of building failure i;
$C_{q,i}$ Costs, indirect made as a result of building failure i;
N Number of inhabitants of one population (group)
$H_n$ Number of dwellings, in which implemented for population N
$\Delta W_{f,i}$.. Difference in Weight- Factors for 2 levels of one disability i;
$P_i$ Period of disability i; (the quotient of prevalence and mortality)
$P_b$.........Implementation Period of the built-intervention
$C_{a,c}$ Annual Costs of one year full 100 % care-services
$P_m$ **P**eriod of precocious mortality; for ageing people set to zero
$C_y$..........Annual **C**osts factor for 1 **y**ear of precocious mortality
$\phi$ Average number of inhabitants per dwelling
g Factor for Implementation Rate of preventive provision in the house stock
m …… Multiplier for the average number of times, persons move home in exploitation period dwelling
Factors F
a...........Appreciation factor for accommodative acceptance of intervention
**d** ……. Evidence of dwelling- related disability

**Scheme 3: The Disability Impact Formula and its components**

## 8 DISCUSSION

The Disability Impact Formula in scheme 3 relates building features at the left side with public-health-items at the right side. Building engineering may consider the public-health side as a given starting point for elaboration of a programme for health-supportive intervention. The Prevalence data in the formula are merely an indication of the actual health-situation at baseline of a preventive programme. The data change as a result of demographic grey growing of the population, its declining vitality status and also by the interventions in the living-conditions.

### 8.1 Consequent split of items, controlled by architecture and by human studies

In the implementation period are to be added 1.the period of diagnosis, 2.the period of planning and installation of a provision and 3.the period of training and individual default setting of a preventive provision. At least the period of planning and installation depends on the way of technical implementation of provisions in the house stock. Large implementation-periods in combination with a short period of beneficial use will result in low efficacy. The implication is that general interventions will be limited to the most common disabilities with highest prevalence and mean period of benefit.

### 8.2 General application versus application strictly for patients

In a given budget however general implementation implies relative low individual investments per dwelling Cp,I. A strategy that focuses on implementation of provisions in 100 percent of dwellings of inhabitants with the disability, allows the highest individual budget per dwelling. The incidental application of preventive provisions in a huge diversity of architectural configurations of the house stock implies a lack of economy of scale and a sub-efficient use of the higher budget. Affordance-theory allows a third strategy. Provide general provisions within the whole house stock that afford a quick incidental optional install of definitive provisions for the prevalence group of a disability.

> I.e. when 10 percent of population suffers a chilly constitution, the provision in the whole house stock of a central heating that generally generates indoor-air temperatures of 23 ºC is overdone.
>
> Individual temperature regulation in a range between 18 º C and 24ºC affords an intervention by setting of new temperature-standards. Required are both sufficient capacity of heating and a device.

### 8.3 The ethics of calculating mortality and morbidity

The Disability Impact Formula focuses on the Quality of life. Public efforts in care-service are compared with building-exploitation. Affordability will enable the structuring of needed economic efforts better than without the formula. The appreciation of life and vitality may largely differ.

### 8.4 Affordability as an economic formula for health-supportive investment

Economic investments in sustainable goods such as building in general are planned for a period of several decades. The break-even point for health-support should be calculated from the average health-situation, which is at least ten to twenty years ahead in future. An extension of demographic trends in health and vitality sometimes is suggested (RIVM 1997) providing the opportunity to compare these figures with exploitation-directives for buildings in for example the year 2015.

## 9 CONCLUSIONS

Simulation of the medical impact of provisions in building constructions, supporting health and vitality of its inhabitants provides is to provide a quick scan for design decisions.

The Disability-Impact Formula is a generalisation of the Torroya formula, that relates building collapse risk and indirect public costs. The developed formula balances the investments in health-preventive provisions in the house stock with a budget, equal to the potentially prevented public costs evoked by disabilities, medication and care-assist. The formula allows a multidisciplinary equitation for one apart disability. For building engineering and public housing the Disability Impact Formula relates variables of housing with a given budget and therefor allows

Further research has to be done towards the evidence of disabilities, expressed as a result of the design of the building. PDAYE-data will enable predictive simulation of the effects of both improved criteria for initial design of the built material context and its implementation-period of interventions in building-constructions. The Disability- Impact Formula for disability I provides a multidisciplinary tool that relates the interests of both the public health domain and the building engineering domain.

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# Performance Test Results Of Strained Bituminous Membranes Under Hydrostatic Pressure

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**Summary: Basement envelope systems are designed as external and internal tanking with bituminous membranes when water that exerts pressure to basement walls and floor slab is the primary concern. One of the performance requirements of the basement envelope system is established, as it must prevent the passage of water under pressure. The component of the system that address the requirement is bituminous membrane; therefore, the performance criteria of the membrane is determined as it must have low permeability and maintain its property during its service life. In actual service life, the satisfactory performance of the tanking with bituminous membrane is achieved if the physical property of the membrane in actual service conditions meets the performance criteria. If not, basement problems occur. In actual service conditions, the membrane is subjected to external agents that strain the membrane excessively. Besides, the strained membrane, under water pressure, may transmit water in vapor form other than liquid form. As the current test methods which determine the resistance of bituminous membranes to water pressure do not involve unstrained membrane samples and a method to measure whether or not membrane transmits water in vapor or liquid form; a performance test method is developed in which the external agents that strain the membrane excessively in external and internal tanking are identified; the values of the strain and the water pressure are put together as the 'performance levels' of the membranes in actual service conditions. Then, the value of the strain, given in the performance levels, is simulated on the membrane sample in laboratory conditions. In the test apparatus, water pressure is exerted on one side of the strained membrane sample in the hydrostatic testing equipment and from the other side of the membrane, the transmission of water either in liquid or vapor form is measured by a measurement system.**

**The aim of this paper is to present the performance test results of strained bituminous membranes under hydrostatic pressure. The method is conducted on oxidized bitumen and APP modified bituminous membranes. In some performance levels, oxidized bitumen and in some performance levels APP modified membrane samples have transmitted water in vapor form, which condensed as liquid water. In conclusion, at each performance level, the membrane types which have not transmitted water in liquid and vapor form are determined as membrane types which meet the criteria and enable the system to fulfill its performance requirement.**

**Keywords: Performance testing, bituminous membranes, hydrostatic pressure, waterproofing**

## 1 INTRODUCTION

Basement is somewhat invisible and the envelope is sometimes ignored component of the building. The primary concern of the designer and the builder is to meet the safety requirement of the user; therefore, the only function of the basement envelope is established as structural adequacy. The user requirements such as health and comfort are ignored or the environmental agents that affect the basement envelope are not fully recognized; thus, less attention is given to separate the basement indoor climate from the outdoor soil climate. As the basement envelope is exposed to below-grade environmental agents, it often fails to respond to them and its performance proves to be unacceptable. Leaky and damp basements, corrosion of the reinforcements, cracked walls, frost-heaved walls, the smell of mould and mildew are some of the major basement problems. On the other hand, the occupants of the buildings expect that the basement indoor climate will provide a prime living space. In order to achieve an effective control of the indoor climate, the occupants take some precautions such as using humidifiers during the winter and dehumidifiers during the summer to maintain an acceptable indoor relative humidity, re-excavating the soil around leaky basements, using extra equipment for heating and cooling and in extreme cases, rebuilding the basement envelope. At the end, the occupants undertake the cost of these precautions; and, they demand that the basement envelope shall provide an effective environmental separation besides structural adequacy. Today's homebuyers want a healthy and energy-efficient home, which they cannot only afford to buy; but also afford to live in. Therefore, the aim is to design and build a basement envelope system, which performs satisfactorily under the effects of the environmental agents and provides livable and usable basement indoor environment that satisfies user requirements. In order to develop a better understanding of the intended role that is to be played by the basement envelope for the people designing and constructing it, the need to establish performance requirements and criteria for the systems and the materials have arisen.

Performance requirements for the basement envelope are developed from the user requirements and performance analysis of the envelope; where the environmental agents that affect the envelope are identified, they are than related to the corresponding properties of the materials of the envelope and the processes involved in the interaction of the materials with their environment are analyzed. Basement envelope involves multiple materials or system of materials which one of them must at least satisfy one requirement of the envelope. After the fundamental performance requirements and the components which address the requirements for the basement envelope are determined, the expected physical property of the material; in other words, performance criteria for the components of the envelope are established. The basement envelope as a system is designed when all the components with the established performance criteria are assembled. In actual service life, the satisfactory performance of the basement envelope system is achieved if the physical property of the material under actual service conditions meets the performance criteria. If not, basement problems occur. In order to find out, whether or not the criteria is met by the physical property of the material under actual service conditions, performance evaluation techniques are developed in which the typical conditions of real use are simulated on the material and the physical property of the material is obtained.

Groundwater is an environmental agent that affects basement walls and floors of new constructed buildings. Hydrostatic pressure can force liquid water into, through and out of the basement walls and floors, cracks and joints as well as exert force onto the walls and floors. Therefore, basement envelope system must prevent the penetration of water under pressure is one of the performance requirements of the envelope system. The component of the system that addresses the requirement is selected as bituminous membranes. The performance criteria of the component are established as it must have low permeability and maintain its property during its service life. The other performance requirement of the envelope system is that it must resist the hydrostatic pressure and the components of the system are structural wall and floor slab. The performance criteria of the structural components are determined as they must resist and transfer loads such that strength of materials and soil bearing capacities are within safety limits. The basement envelope system is designed when bituminous membrane, structural wall and floor slab are put together as, external and internal tanking.

In actual service conditions, bituminous membranes are subjected to mechanical agents such as lateral soil pressure, compressive pressure under the floor slab, etc. Concisely, the external agents strain the bituminous membranes excessively. And meanwhile, they are exposed to water pressure. When strained membranes in tanking systems are exposed to water pressure, liquid water may penetrate into, through and out of the membrane if the membrane has micro, capillary or macro pores. If not, water penetration does not occur. It is generally accepted that when the membrane does not permit the passage of liquid water, it is impermeable; or, when it permits the passage of liquid water, it is permeable. On the other hand, when one side of the strained membrane is subjected to liquid water under pressure, liquid water may evaporate into the molecular pores in the membrane as a result of vapor pressure differential. The membrane will permit the passage of water vapor if the size of the molecular pores is large than the size of the water molecule. Therefore, the performance criteria is not attained; in other words, the membrane is permeable when it permits the passage of water vapor, as basement problems such as reinforcing steel corrosion, condensation of water vapor at the inside or at the inner surface depending on the temperature and the relative humidity of the structural components can occur.

Current standards and general performance documents comprising the test methods for determining the hydrostatic pressure resistance of bituminous membranes are considered to be testing for quality control, not for testing to predict in-service performance of bituminous membranes in tanking systems. In these test methods, unstrained membranes are used, the leakage of water is observed by naked eye and a method to measure whether or not the membrane permits the passage of water vapor under water pressure is not involved. Besides, test time is insufficient to represent the in-service life of the membrane. As a consequence, a performance test method is developed to predict in-service performance of bituminous membranes in tanking systems. The performance test method comprises two stages. Initially, the performance levels of bituminous membranes in actual service conditions are determined. Then, in the test method, each performance level is simulated on the membrane sample and the permeability property of the sample is measured in the testing apparatus experimentally, (Sahal & Ozkan, 1999).

# 2 PERFORMANCE TEST METHOD

## 2.1 Performance levels of bituminous membranes in tanking systems

In the initial step of the method, the mechanical agents that strain the membrane excessively in external and internal tanking are identified. The highest value of stress which the membrane undergo due to one of the mechanical agents and the value of the hydrostatic pressure which the strained membrane is exposed simultaneously are calculated for a single story high basement. And, both values are assembled as one of the performance levels of bituminous membranes in actual service conditions. The calculation is repeated for a two, three and eight story high basement for the same mechanical agent, Table 1.

The membrane, which is applied to the outer surface of the basement walls, is subjected to mechanical agent, which is lateral soil pressure. As lateral soil pressure increases linearly with depth, the membrane is exposed to unequal values of compressive pressure; thus, the visco-elastic bitumen deforms. According to the Coulomb theory, the highest value of pressure that is exerted on the membrane in a single, two, three and eight story high basements are calculated as 0.01, 0.02, 0.03 and 0.1 $N/mm^2$, respectively. The values of hydrostatic pressure in a single, two, three and eight story high basement are calculated as 0.02, 0.05, 0.10 and 0.25 MPa, respectively. In Table 1, performance levels 1.1, 1.2, 1.3 and 1.4 represent the highest values of stress due to the lateral pressure and the values of hydrostatic pressure which the membrane is subjected in a single, two, three and eight story high basement, respectively.

**Table 1. Performance levels**

| *Performance Levels* | *Mechanical Agents* | *Stress* | | | *Pressure, (MPa)* | |
|---|---|---|---|---|---|---|
| | | *Compressive Pressure ($N/mm^2$)* | *Tensile Stress (N/50 mm.)* | *Crack (mm.)* | *Air* | *Water* |
| 1.1 | Lateral soil pressure | 0.01 (0) | | | | 0.02 |
| 1.2 | | 0.02 (0) | | | | 0.05 |
| 1.3 | | 0.03 (0) | | | | 0.10 |
| 1.4 | | 0.1 | | | | 0.25 |
| 2.1 | Compressive pressure | 0.1 | | | | 0.02 |
| 2.2 | | 0.2 | | | | 0.05 |
| 2.3 | | 0.3 | | | | 0.10 |
| 2.4 | | 0.5 | | | | 0.25 |
| 2.5 | | 0.8 | | | | 0.25 |
| 3.1 | Compressive pressure and air pressure | 0.1 | | | 0.02 | 0.02 |
| 3.2 | | 0.2 | | | 0.05 | 0.05 |
| 3.3 | | 0.3 | | | 0.10 | 0.10 |
| 3.4 | | 0.5 | | | 0.25 | 0.25 |
| 3.5 | | 0.8 | | | 0.25 | 0.25 |
| 4.1 | Compressive pressure and tensile stress | 0.1 | 100, 200...n | | | 0.02 |
| 4.2 | | 0.2 | 100, 200...n | | | 0.05 |
| 4.3 | | 0.3 | 100, 200...n | | | 0.10 |
| 4.4 | | 0.5 | 100, 200...n | | | 0.25 |

The membrane that takes place between the structural floor slab and the concrete blinding remains under compressive pressure due to the building loads. The base pressure distribution under the floor slab is not equal; in other words, the pressure is higher under the column bases; therefore, the bitumen is subjected to unequal values of pressure and as a result it deforms. As the compressive pressure must be lower or at least equal to the safety bearing capacity factor of the saturated soil, these values are accepted as the values of compressive pressure that strain the membrane. The values of stress in a single, two, and three high basements are accepted as 0.1, 0.2, and 0.3 $N/mm^2$, respectively and for an eight story high basement as 0.5 and 0.8 $N/mm^2$. The values of hydrostatic pressure in a single, two, three and eight story high basement are calculated as 0.02, 0.05, 0.10 and 0.25 MPa, respectively. The values of stress due to the compressive pressure under the structural floor slab and the values hydrostatic pressure in a single, two and three story high basement are presented in Table 1 as performance levels 2.1, 2.2, and 2.3, respectively. For an eight story high basement, performance levels of 2.4 and 2.5 are required.

Besides compressive pressure, the membrane is subjected to air pressure. After the construction, when the pumping out of the groundwater is stopped, the rising water will fill the air pockets and push the air to the membrane. The air under may penetrate through the membrane providing easy access for the water afterwards. The values of air pressure that affect the membrane in a single, two, and three and eight story high basement are accepted as 0.02, 0.05, 0.10 and 0.25 MPa, respectively, Table 1.

The steel reinforcement of the reinforced concrete structural slab meets the tensile stress generated in the slab due to the building loads; therefore, it deforms. As the membrane remains under the structural floor slab, it is also subjected to the tensile stresses of the floor slab; in other words, it follows the structural deformation. In Table 1, performance level 4.1, 4.2, 4.3 and 4.4 represents the values of stress due to the compressive pressure and tensile stress as well as the value of hydrostatic pressure in a single, two, three and eight story high basement, respectively.

### 2.2 Test method

In the test method, the values of the stress, given in the performance levels, are simulated on the membrane sample in laboratory conditions. Then, strained membrane sample is placed in the testing apparatus. In the testing apparatus, the value of hydrostatic pressure is simulated on one side of the membrane sample. And, from the other side of the sample, whether or not it permits the passage of water in either liquid or vapor form under water pressure is measured.

#### *2.2.1* Simulation

In order to simulate the lateral soil pressure and compressive pressure on the membrane sample; the sample is cut from the roll and placed between the two steel plates. The two steel plates with the membrane sample between are then placed between the two jaws of the universal compressive strength testing machine. The test sample is loaded to a predetermined point of compressive pressure. Air pressure is exerted on the membrane sample for 24 hours when the strained membrane sample is placed in the testing apparatus. In order to simulate the effect of tensile stress on the membrane sample, the sample is placed between the two jaws of the universal tensile strength testing machine. Then, the sample is loaded to a predetermined point of tensile stress.

#### 2.2.2 Testing Apparatus

The testing apparatus comprises a hydrostatic testing equipment and a measurement system. The hydrostatic testing equipment includes a chamber, a rubber gasket, a steel plate, a clamping bracket, and fasteners to form the completed assembly. The strained membrane sample takes place between the rubber gasket and the steel plate. The function of the chamber is to provide rooms in which water pressure will be build up. Rubber gasket takes place between the front surface of the chamber and the strained membrane sample. Its function is to prevent the leakage of water under pressure between the chamber and the test sample. The function of the steel plate is to provide an environment in which water in liquid or vapor form that may pass through the test sample is collected when the sample is exposed to water under pressure. Therefore, there is a gap in the body of the steel plate. The surface of the steel plate that receives the membrane has a rectangular hole, which opens to the gap in the body of the plate. The other surface of the steel plate has a circular hole, which also opens to the gap. The measurement system comprises an absolute humidity sensor, a signal conditioning circuitry, a universal multimeter and a PC.

The operation of the testing apparatus: The test apparatus is situated in a relative humidity and temperature-stable room. In order to conduct a test, the rubber gasket is set against the chamber, and then the strained membrane sample is placed. The steel plate and steel bracket are set against the membrane sample. The fasteners are tightened, clamping the membrane to the chamber gradually to form the completed assembly. Then, the absolute humidity sensor is mounted to the circular hole at the steel plate.

When one side of the sample is exposed to water pressure, the water in either liquid or vapor form that may pass through the sample is collected in the gap. The gap is sealed to prevent the access of water vapor into the gap from the outer environment or out of the gap to the environment. Meanwhile, the absolute humidity sensor measures the absolute humidity value ($g/m^3$) of the gap and the readings are recorded in the PC. The measurement is conducted per minute for 96 hours. Then, the hourly mean absolute humidity values in the gap are calculated. The maximum absolute humidity value in the gap; in other words, the absolute humidity value in which the condensation begins to occur in the gap is also calculated. Both the hourly mean absolute humidity values and the maximum absolute humidity value in the gap are presented in a graphic with the test time.

## 3 PERFORMANCE TESTING

### 3.1 Bituminous membranes used in testing

Bituminous membranes that are used in testing are single ply oxidized bitumen with unwoven glass fibre (60 $g/m^2$) with a thickness of 2.4 mm., oxidized bitumen with woven glass fibre (200 $g/m^2$) with a thickness of 2.9 mm. and APP modified bitumen with polyester reinforcement (180 $g/m^2$) with a thickness of 3 mm.

### 3.2 Performance levels applied to bituminous membranes in testing

Performance level 1.3 is applied to bituminous membranes, which are oxidized bitumen with unwoven and woven glass fibre reinforcement and APP modified bitumen with polyester reinforcement. Performance level 1.4 is applied to oxidized bitumen with woven glass fibre reinforcement and APP modified bitumen with polyester reinforcement. Performance levels 2.4, 2.5, 3.4, 3.5 and 4.4 are applied to APP modified bitumen with polyester reinforcement, only.

### 3.3 Test results

When single-ply oxidized bitumen with unwoven glass fibre reinforcement was strained with a compressive pressure of 0.03 (0) $N/mm^2$ and 0.10 MPa of water pressure was exerted on one side of the membrane sample (performance level 1.3), the initial amount of absolute humidity in the gap has increased within the first hour of the test and it has reached maximum absolute humidity at the end of fifth hour, Fig. 1. In other words, the membrane sample has permitted the passage of water vapor within the first hour of the test, the amount of water vapor in the gap has increased in the following hours and it has condensed at the end of fifth hour. It is concluded that the membrane is permeable as it permits the passage of water vapor and water vapor condenses as liquid water; therefore, the membrane can not meet the performance criteria in the specific actual service conditions. On the other hand, oxidized bitumen with woven glass fibre reinforcement and APP modified bitumen with polyester reinforcement have not permitted the passage of water in either liquid or vapor form at the same conditions as the initial amount absolute humidity in the gap has remained unchanged during 96 hours of the test time, Figs 2 and 3. Thus, the performance criteria have been attained by these membrane types in the same specific actual service conditions.

The permeability property of single-ply oxidized bitumen with woven glass fibre reinforcement and APP modified bitumen with polyester reinforcement when they are strained with a compressive pressure of 0.1 N/mm$^2$ and they are exposed to 0.25 MPa of water pressure, are presented in Figure 4 and Figure 5, respectively. It can be concluded that the membranes are impermeable, as the absolute humidity values of the gap have remained unchanged within 96 hours of the test time.

When performance levels 2.4, 2.5 and 3.4 have been applied to single-ply APP modified bitumen with polyester reinforcement, the membrane has not permitted the passage of water in either liquid or vapor form, Figs 6 and 7. On the other hand, when performance level 3.5 was applied to the membrane, it has permitted the passage of water vapor at the end of 77$^{th}$ hour. The water vapor has condensed as liquid water at the end of 84$^{th}$ hour, Fig. 8. In other words, when the membrane was strained at 0.8 N/mm$^2$ of compressive pressure and it was subjected to 0.25 MPa of water pressure for 96 hours, it has proved to be impermeable; but, when the membrane was strained at 0.8 N/mm$^2$ of compressive pressure and it was subjected to 0.25 MPa of air pressure for 24 hours and then 0.25 MPa of water pressure for 77 hours, it has proved to be impermeable.

In performance level 4.4, single-ply APP modified bitumen with polyester reinforcement was strained at 0.5 N/mm$^2$ of compressive pressure and 300 N/50 mm. tensile stress and it was subjected to 0.25 MPa of water pressure. The initial absolute humidity value of the gap has started increasing at the end of 2$^{nd}$ hour and it has reached to the maximum absolute humidity of the gap at the end of 7$^{th}$ hour, Fig. 9. The membrane has also permitted the passage of water vapor at the end of 43$^{rd}$ hour which has condensed at the end of 47$^{th}$ hour when it was strained at 0.5 N/mm$^2$ of compressive pressure and 200 N/50 mm. tensile stress and it was subjected to 0.25 MPa of water pressure, Fig. 10. Similar result was obtained from the test when the membrane was strained at 0.5 N/mm$^2$ of compressive pressure and 100 N/50 mm. tensile stress and it was subjected to 0.25 MPa of water pressure, Fig. 11. The membrane has permitted the passage of water vapor at the end of 63$^{rd}$ hour and condensation has occurred at the end of 68$^{th}$ hour. The time when the membrane permits the passage of water vapor increases as the value of tensile stress applied to the membrane decreases.

## 4 CONCLUSION

A performance test method has been developed to predict in-service performance of bituminous membranes in tanking systems. The test results proved that in some performance levels, some of the membrane samples have permitted the passage of water vapor and water vapor has condensed and has been released as water in liquid form. The performance levels in which the membrane samples have transmitted water vapor is as follows:

- Oxidized bitumen membrane with unwoven glass fabric reinforcement – 0.03 N/mm$^2$ compressive pressure, 0.10 MPa water pressure,
- APP modified bituminous membrane with polyester mat reinforcement - 0.8 N/mm$^2$ compressive pressure, 0.25 MPa air pressure and water pressure,
- APP modified bituminous membrane with polyester mat reinforcement - 0.5 N/mm$^2$ compressive pressure, 300 N/ mm. tensile stress, 0.25 MPa water pressure,
- APP modified bituminous membrane with polyester mat reinforcement - 0.5 N/mm$^2$ compressive pressure, 200 N/ mm. tensile stress, 0.25 MPa water pressure,
- APP modified bituminous membrane with polyester mat reinforcement - 0.5 N/mm$^2$ compressive pressure, 100 N/ mm. tensile stress, and 0.25 MPa water pressure.

In the remaining performance levels, the membrane samples have not permitted the passage of water in either liquid or vapor form. This points out that bituminous membranes do not permit the passage of water in liquid form unless there has been a defect in the membrane during the application process or the production stage.

When testing is conducted in all types of the bituminous membranes that are found commercially, the membrane types that can be used in external and internal tanking depending on their permeability property and the performance levels can be determined.

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## 6 APPENDIX

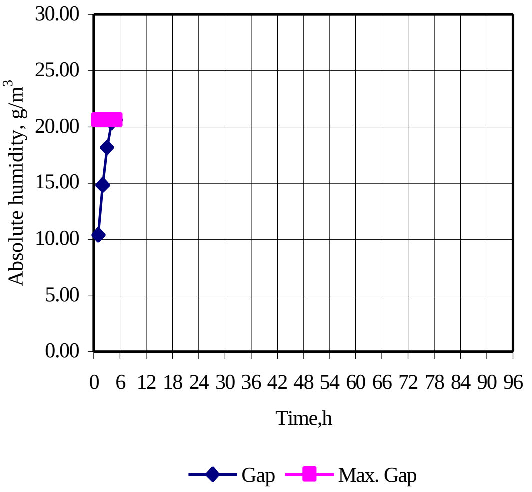


**Figure 1. Performance of single-ply oxidized bitumen with unwoven glass fibre reinforcement at performance level 1.3.**

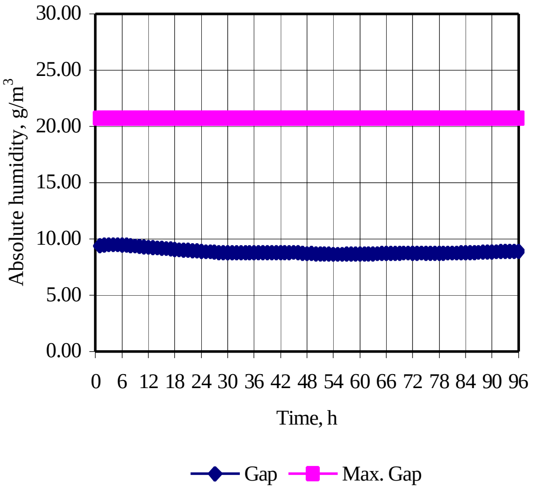


**Figure 2. Performance of single-ply oxidized bitumen with woven glass fibre reinforcement at performance level 1.3.**

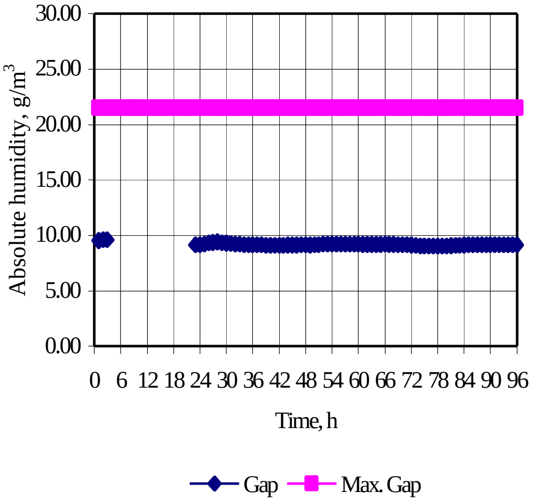


**Figure 3. Performance of APP modified bitumen with polyester reinforcement at performance level 1.3.**

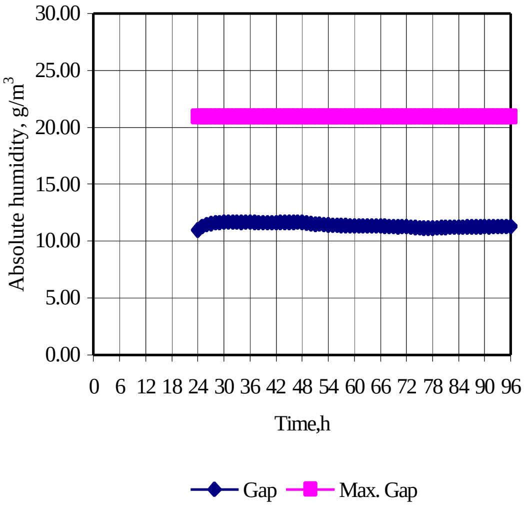


**Figure 4. Performance of oxidized bitumen with woven glass fibre reinforcement at performance level 1.4.**

# Controlling Indoor Air Pollution By Product Labels For Emissions From Building Materials And Contents

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**Summary: It has been a common research finding that most indoor air pollutants arise predominantly from the building materials, contents and appliances used in buildings, rather than the ingress of outdoor urban air pollutants. Many of these pollutants are toxic and, since indoor air levels can much exceed outdoor levels, and modern populations typically spend over 90% of their time indoors, there is a need to assess occupant exposures and control those exposures without unacceptable risks.**

**Controlling emissions from pollutant sources is considered to be the optimum approach for indoor pollution control. This requires an understanding of how these materials emit pollutants (pollutant type, quantity and persistence), the risks presented from exposure to the pollutants, and how control strategies (voluntary labels, regulations) can be implemented. This paper discusses the research and technologies that have developed over the last five years for controlling pollutant emissions, and their implementation in different countries.**

**Keywords: Indoor air, pollutant emissions, building materials, labels, control.**

## 1 INTRODUCTION

There have been many studies into air pollutant levels within buildings (Spengler & Samet 1991; Brown 1997; 1999a) that have shown that pollutant levels are usually much higher indoors than outdoors, generally because of specifically identified sources of the pollutants (e.g. nitrogen dioxide in the 600,000 Australian households with unflued gas heating, volatile organic compounds (VOCs) in the 120,000 new Australian buildings occupied each year). This is best demonstrated by the ratios of indoor to outdoor pollutant concentrations which are typically:

- VOCs in established buildings 8:1.
- VOCs in new buildings >200:1.
- Formaldehyde in conventional buildings 10:1.
- Formaldehyde in mobile buildings 50:1.
- Nitrogen dioxide in house with unflued gas heating ~5:1.
- Respirable particles in buildings without smoking 0.1–1:1 (i.e. outdoor particles predominate).
- Respirable particles in buildings with smoking >10:1 (i.e. indoor particles predominate).

In many of these cases, the indoor air concentrations exceed health-based environmental exposure goals (usually applied by environment agencies to outdoor air) by significant margins. Also, it is known that people in developed countries spend the major proportion of their lives indoors rather than outdoors. In Australia, it has been estimated from ABS survey (Newton *et al*. 2001) that on average we spend 90% of our time indoors, 7% in transit (usually by car) and only 3% outdoors. Pollutant exposure is estimated from:

$$Exposure = Concentration \times Time\ exposed \qquad (1)$$

and so for many pollutants, population exposure is dominated by indoor air pollutants. Three approaches are possible to reduce indoor air pollutant exposures:

- Increased building ventilation.
- Inclusion of air-cleaning devices in buildings.
- Reductions of pollutant emissions from the source materials.

Increased ventilation rates will be effective in reducing all indoor air pollutants, whatever their sources. However, ventilation code requirements have been historically set at minimum levels to remove pollutants and moisture from occupants, in order to prevent building air being perceived as ‘stuffy’. These minimum levels can be increased, but the degree of increase needed will depend on the exposure reduction necessary for the most toxic of the pollutants, not only an unknown factor, but often requiring impractical ventilation rates to achieve an acceptable reduction. An added problem with this approach is the increased energy consumption (and related environmental impacts) that will flow on from using higher ventilation rates.

Including air-cleaning devices in buildings is a feasible strategy that has had limited use in mechanically ventilated buildings for many decades, especially for the removal of large particles capable of fouling air-conditioning components. However, the performance and design requirements for these, both in mechanically and naturally ventilated buildings, still remains an area of technical development (Brown 1999b). Also, the approach will require regulation of the maintenance of these devices if they are to remain effective.

Control of pollutant emissions from source materials is considered to be the optimum strategy for the control of indoor air pollution. This approach allows:

- Adherence to the ventilation practices used to date, without impact on energy conservation measures.
- Identification and control of the major sources of targetted pollutants, where these sources are shown to lead to unacceptable toxic pollutant exposures.

However, pollutant emission control requires that these materials be selected according to the type, quantity and persistence of toxic air pollutants, with this measurement being predictive of low-polluting performance in buildings. This paper will discuss the measurement of these properties, the rating of pollutant emissions, and the selection (or product labelling) of low-polluting building materials.

## 2 MEASURING POLLUTANT EMISSIONS FROM MATERIALS

Indoor air pollutant emissions of materials and appliances are measured in simulated building environments using ‘dynamic environmental chamber’ technology, which was initiated in research studies from the late 1980s (Tichenor 1987; Tichenor & Guo 1987; Sanchez *et al*. 1987; Dunn 1987; Dunn & Tichenor 1988) and has become increasingly sophisticated (ASTM 1997; Mason *et al*. 1999; Zhang *et al*. 1999). In this technology, materials are tested under conditions where:

- Air environmental conditions (temperature, humidity, ventilation rate, air velocity) are tightly controlled to the conditions expected in buildings (typically 23°C, 50% RH, 0.5 or 1.0 air changes per hour, 0.1–0.3 m/s, respectively).
- Chambers are constructed of non-emitting materials, such as glass and stainless steel, with little or no ‘sink’ effects (adsorption/desorption processes) for air pollutants and are supplied with purified air, these features ensuring that any pollutants measured in the chamber air originate only from the test material.
- The quantity of material measured in the chamber relative to chamber volume (the product loading ratio) is similar to that which is typically used in buildings (e.g. flooring and floor coverings 0.4 $m^2/m^3$, paint 0.5 $m^2/m^3$) in order to simulate how the material would emit pollutants in practice.
- The air in the chamber is well mixed, so that the pollutant concentration in the chamber air ($C$ $\mu g/m^3$) is directly related to the pollutant emission rate of the material ($R$ $\mu g/h$) by:

$$R = C.V.N \qquad (2)$$

where $V$ is the chamber volume ($m^3$); and $N$ is the chamber ventilation rate ($h^{-1}$).

Dynamic environmental chamber testing allows a material property called the Emission Factor (*EF*) to be estimated from *R*, whereby the pollutant emission rate relative to the quantity of material is estimated, e.g. relative to area ($\mu g/m^2.h$) or item ($\mu$g/h.workstation) or process operation such as gas combustion (ng/J) or printing ($\mu$g/copy). Note that *EF* may be constant for some products (e.g. office equipment, gas appliances) or variable over time (which it is for most manufactured products) according to specific *source emission models* (Guo 1996). Emission factors for many pollutants vary rapidly over time, particularly wet products such as paints, adhesives and sealants. The value of an *EF* source emission model is that it can be used to model indoor air scenarios and estimate indoor pollutant concentrations for comparison with health-based exposure goals (see later).

Recent CSIRO studies on pollutant source emission models have been published for paints (Brown 1998a), wood-based panels (Brown 1999c), carpets (Brown 2001a), office equipment (Brown 1999d) and gas appliances (Brown *et al*. 2000). A range of source emission models have been identified, as follows.

### 2.1 Constant emission rate

A constant emission source emits pollutant at a constant rate per unit of product. An unflued gas heater was found to emit nitrogen dioxide or carbon monoxide at constant rates per unit of energy consumed, i.e. its *EF* on the basis of ng/J was constant, and so with information on the heating rate of the heater, the pollutant emission per time could be estimated. A photocopier was found to emit VOCs and respirable particles at constant rates per copy produced, and so pollutant emission per time could be estimated using the rate of operation of the copier.

## 2.2 First-order emission decay

First order emission decay occurs to the following source model:

$$EF_t = k_1\, M_o \exp(-k_1 t) \qquad (3)$$

where $EF_t$ is the emission factor at time $t$; $k_1$ is the first-order rate constant for emission decay; and $M_o$ is the mass of pollutant in the source at time $t = 0$ (note that $EF_o = k_1\, M_o$).

Thus, this model is fully defined by the parameters $k_1$ and $M_o$. Interestingly, this model has been found to be applicable to the emission decays of many materials, especially wet and semi-dry materials, as summarised in Table 1. These model parameters can be used to estimate pollutant concentrations in buildings that use these materials. They can also be used to make broad comparisons between different types of materials; for example, the data in Table 1 shows that:

- Each acrylic paint (A and B) has its own emission property, and both a low-odour acrylic paint (largely emitting propylene glycol) or a ‘natural’ paint emit large quantities of pollutants.
- A zero-VOC acrylic paint exhibits changes by orders of magnitude in both the quantities emitted and their rates of decay.

**Table 1. First order emission decay parameters for materials**

| *Material* | *Pollutant* | *$k_1$ ($h^{-1}$)* | *$M_o$ ($mg/m^2$)* |
|---|---|---|---|
| Acrylic paint A | Texanol® | 0.11 | 790 |
| Acrylic paint B | TVOC | 0.13 | 1800 |
| Low-odour acrylic paint | Texanol® | 0.08 | 260 |
| Zero-VOC acrylic paint | TVOC | 0.11 | 730 |
| Enamel paint | Propylene glycol | 0.13 | 2400 |
| ‘Natural’ paint | TVOC | 0.11 | (730) |
| Wool carpet (new) | 2-Butoxyethanol | 3.6 | 3.6 |
| | TVOC | 2.3 | 24 |
| Particleboard (new) | Xylene isomers | 1.6 | 1400 |
| | TVOC | 0.38 | 20,000 |
| MDF (new) | Limonene | 1.2 | 470 |
| | TVOC | 1.3 | 870 |
| | 4-Phenylcyclohexene | 0.003 | 28 |
| | Styrene | 0.068 | 18 |
| | TVOC | 0.051 | 58 |
| | Formaldehyde | 0.001 | 380 |
| | Methanol | 0.006 | 520 |
| | Hexanal | 0.0002 | 260 |
| | TVOC | 0.014 | (84) |
| | Formaldehyde | 0.0006 | 650 |

- An enamel paint emits an order of magnitude higher quantities of VOCs, but with higher decay rates.
- Wool carpet is a much lower reservoir of VOCs than interior paints (except zero-VOC paint), but its decay constants can be an order of magnitude lower, leading to sustained emissions.
- Particleboard emits similar quantities of VOCs as acrylic paints, but its emissions are much more sustained due to the much smaller decay constants.
- Formaldehyde emissions from particleboard and MDF decay at very small rates and will be sustained for very long periods (estimated as many months to years).

### 2.3 Double-exponential emission decay

This describes emission by a material that effectively acts as two concurrent first-order sources, each differing markedly in decay constant:

$$EF_t = k_1\, M_{o1} \exp(-k_1 t) + k_2\, M_{o2} \exp(-k_2 t) \qquad (4)$$

Formaldehyde emissions from wood-based products over periods of months to years have been found to emit by this model, as shown for moisture resistant-grade particleboard in Fig. 1. Note that for such products, the very small values for $k_2$ (0.000001 $h^{-1}$ in the case of particleboard) result in long-term emissions over periods of years (probably for the life of the products), and this has been observed to occur in buildings using such materials (Brown 1998b; 2001b).

## 3 RATING OF POLLUTANT EMISSIONS FROM MATERIALS

The rating of pollutant emissions from materials requires:

- Identification of the presence of those pollutants of significance to occupants from all health perspectives, e.g. malodour, eye/nose/throat irritation, allergenicity, systemic toxicity, reproductive toxicity and cancer.
- Determining the appropriate exposure guidelines for these pollutants.
- Estimating whether these guidelines would be exceeded during the occupancy of buildings.

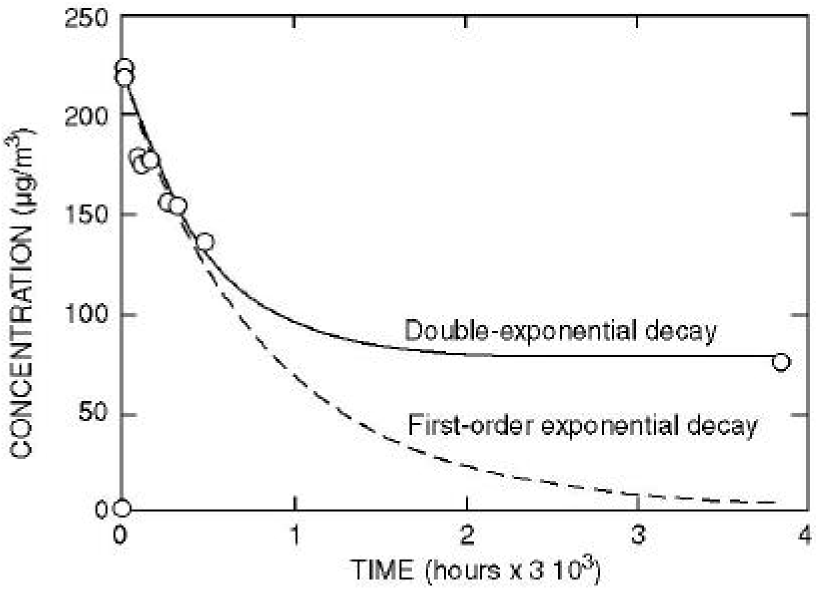


**Figure 1. Formaldehyde emission from particleboard in a room chamber over a period of six months, with comparison of a first-order exponential decay model up to 20 days and a double-exponential decay model for full period.**

There are many sources of information on pollutant toxicity and exposure guidelines that assist in meeting the first two requirements. Occupational exposure guidelines are not generally used because they have been derived for different exposure scenarios than indoor air (e.g. they apply to the 40 hour per week exposure of workers rather than the nearly continuous exposure of indoor air, and they are not designed to protect the more susceptible sectors of the population). In Australia, the NHMRC has recommended indoor air guidelines (Brown 1997) as follows (all concentrations expressed at 0°C and 101 kPa):

- Formaldehyde 130 $\mu g/m^3$ (ceiling limit).
- Total VOC (TVOC) 500 $\mu g/m^3$ and any VOC 250 $\mu g/m^3$ (one-hour average).
- Carbon monoxide 9 ppm (eight-hour average).
- Ozone 260 $\mu g/m^3$ (one-hour average).

However, more detailed air quality guidelines have been recommended by the World Health Organisation (1999), as presented in Table 2.

Estimating whether these guidelines are exceeded where products that emit such pollutants are used in buildings, requires application of indoor air models (Sparks 1996; Guo 1996) with the source emission models described above, and assumptions about certain properties of the building, specifically:

- The quantity of material used – generally known.
- The natural ventilation rate of the building – generally unknown, but reasonable minimum estimates for Australian buildings would be a long-term average of one air change per hour (ACH) and a short-term average of 0.5 ACH.

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- The mechanical ventilation rate of the building – generally known at construction.
- The 'sink' effects of interior surfaces for air pollutants, whereby a proportion are removed from air in the short term to be slowly re-emitted afterwards – can be estimated, but if ignored it can provide a safety margin (overestimate) for acute exposure and a possible underestimate for chronic exposure depending on sink characteristics of interior surfaces
- The distribution of air movements within a building – generally unknown, though pollutant measurements in buildings suggest they are generally well mixed (Brown 2001a).

**Table 2. WHO air quality guidelines for Europe**

| VOC | Guideline ($\mu g/m^3$) | Averaging time |
|---|---|---|
| | Lifetime cancer risks at 1 $\mu g/m^3$ | |
| Acetaldehyde[a] | $(1.5–9) \times 10^{-7}$ | |
| Acrylonitrile[a] | $2 \times 10^{-5}$ | |
| Benzene[a] | $4 \times 10^{-6}$ | |
| Chloroform[a] | $4 \times 10^{-7}$ | |
| 1,2-Dichloroethane[a] | $(0.5–2.8) \times 10^{-6}$ | |
| 1,1,2,2-Tetrachloroethane[a] | $(0.6–3.0) \times 10^{-6}$ | |
| Trichloroethylene[a] | $4.3 \times 10^{-7}$ | |
| Vinylchloride[a] | $1 \times 10^{-6}$ | |
| Acrylic acid | | |
| 2-Butoxyethanol | 54 | 1 year |
| Carbon tetrachloride | 13,000 | 1 week |
| Chlorobenzene | 6 | 1 year |
| 1,4-Dichlorobenzene | 71 | 1 year |
| Dichloromethane | 134 | 1 year |
| Ethylbenzene | 3000 | 24 hours |
| Formaldehyde | 22,000 | 1 year |
| Methyl methacrylate | 100 | 30 minutes |
| Styrene | 200 | 1 year |
| Styrene[b] | 260 | 1 week |
| Tetrachloroethylene | 7 | 30 minutes |
| Toluene | 250 | 24 hours |
| 1,3,5-Trichlorobenzene | 260 | 1 week |
| 1,2,4-Trichlorobenzene | 36 | 1 year |
| Xylene isomers | 8 | 1 year |
| | 870 | 1 year |

[a] Carcinogens.

[b] Based on sensory effects or annoyance reactions.

Such estimates can be made for individual materials on a case-by-case basis, and this approach has now been formalised for building specification in the Netherlands by its Building Decree. A simpler approach is to estimate acceptable pollutant emission criteria for a product type, based on its typical usage pattern, and to label materials that meet the acceptable criteria as 'low polluting'. This approach is now becoming widely used in many countries.

## 4 PRODUCT LABELLING FOR POLLUTANT EMISSIONS

Indoor air labelling of products for their pollutant emissions has three basic requirements:

- Assessment methods must simulate the end-use conditions of the products.
- Pollutant emission limits should be expressed as emission factors or indoor pollutant concentrations estimated from *EF*s, in both cases relative to health-based exposure goals for air pollutants.
- Due to the change in product emissions over time, the pollutant impacts on building occupants will depend on the rates of emission decay and the delay expected between product installation and building occupancy; hence, any pollutant emission criteria must specify the product age at which it applies, and this age should simulate the above delay. For example, no delay is relevant for emission rates of process operations such as gas heating or photocopier operation (but the rate of the process will need to be specified). The delay between painting and occupancy may be as little as several hours for renovation works or as much as several days for new buildings. Manufactured items such as carpets and wood-based panels may be stored for several days or more before installation into buildings.

Currently, there are many product labelling schemes operating in Europe and North America, and these differ in the degree to which they meet these requirements. Several of these schemes will be discussed here.

In Denmark and Norway, an Indoor Climate Labelling (ICL) scheme (http://www.uk.teknologisk.dk/1689) is operated voluntarily, though Building Codes in both countries have recommended its use, and standard test methods have been developed for a wide range of materials (paints, carpets, partitions, wall systems, flooring, furniture, cabinets). The criteria for the label are the chemical emissions and the odour/irritancy properties of the new products after manufacture. VOC emissions over time are measured in a standard test chamber procedure and are then converted into VOC concentrations in a 'standard room' by a simple model. These VOC concentrations are compared to known odour and mucosal irritation thresholds for VOCs, and the times for the concentrations to decrease to 50% of threshold values are estimated. The label consists of an 'indoor-relevant time value', which is the time required for all VOCs to decrease to 50% of the threshold, i.e. it is the longest of the times determined in the above procedure. Alternatively expressed, it is the time after which an installed product is unlikely to cause odour or musosal irritation to occupants. An odour measurement of emissions using a trained panel is also required at this time.

In Finland, the Ministry of Environment operate a voluntary scheme called 'Classification of Indoor Climate, Construction, and Finishing Materials' (http://www.rts.fi/M1classified.htm) to supplement the building code. It applies to a wide range of product types (wall and ceiling panels, floor coverings, insulations, paints, glues) which are assessed in a standard chamber for odour, for emission factors at 28 days after manufacture, which must not exceed the following limits:

- Odour dissatisfaction <15%.
- TVOC <200 $\mu g/m^2.h$.
- Formaldehyde <50 $\mu g/m^2.h$.
- Ammonia <30 $\mu g/m^2.h$.
- Compounds listed as Category 1 carcinogens, total <5 $\mu g/m^2.h$.

Over 300 products now meet this low-emission criteria, which is approximately 20% of available building materials in Finland.

In Germany, the Federal Environment Agency (UBA) has operated a widely based ecological label since 1977 (http://www.blauer-engel.de), with over 4500 products from 800 manufacturers now using the label. Indoor air pollutant emissions are tailored to the specific products and are integrated with other environmental targets (resource use, recycling, global impact) for low-pollutant paints and varnishes, wood and wood-based products, furniture, gas cookers, copiers and printers. Currently, UBA has formed a 'Committee on the health-related evaluation of building products' which is developing an enhanced assessment scheme for product emissions.

In the USA, a large number of product test standards have been developed by the American Society for Testing and Material (ASTM), and a recent initiative by the Underwriters Laboratories Inc will lead to health-based consensus standards (test methods and emission criteria) for residential and commercial buildings (http://www.ul.com/eph/iaq/index.htm ). The main voluntary industry initiative is the Carpet and Rug Institute's 'Green Label' program for carpet, underlay and adhesive, based on emissions of specific VOCs, TVOC and formaldehyde from each product (http://www.carpet-rug.com).

## 5 CONCLUDING REMARKS

There has been much progress in the control of indoor air pollution through understanding and assessing emissions from manufactured products, and their influence on the health and well-being of building occupants. This has seen the implementation of schemes to limit product emissions. Generally, these have been voluntary schemes which have been loosely referred to in some building codes. A more rigorous approach – the implementation of specific code requirements – is another possiblity and has appeared recently in the Netherlands Building Decree. It is clear that the way that indoor air quality is specified and controlled is changing rapidly, with a central focus on a logical approach to controlling pollutant emissions at source.

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# Prognosis Of Concrete Corrosion Due To Acid Attack

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**Summary: A model is introduced for the prediction of the corrosion of concrete under acid attack at pH values between 4.0 and 6.5. The corrosion process is described by the diffusion of acid in the pore solution coupled to the reaction kinetics between the acid and the different hydration products. The effect of changing porosity is included. The input parameters are measured by exposing mortar containing quartzitic sand to acetic acid buffer solutions. An instationary method is used to determine the diffusion coefficients of the ions including $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ in the pore solution of corroded mortar. The rate constants for the dissolution of $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ contained in the solid phases are determined in solubility experiments. Experiments at pH 5.0 show that the dissolution of phases containing iron and aluminium is slower than C-S-H and calcium hydroxide. This implies that the iron and aluminium phases reduce the speed of corrosion, enhancing concrete durability.**

**Keywords: Concrete, durability, corrosion, acid attack, model**

## 1 INTRODUCTION

The degradation of concrete occurring when the unprotected concrete surfaces of sewer pipes, waste water treatment plants, cooling towers, animal houses etc. are subjected to aggressive acid solutions seriously limits service life. The resistance of concrete against acid attack depends on the concrete composition, the type of aggregate as well as the pH and type of the acid. The modelling of the chemical reactions and ion transport processes leading to corrosion is highly important for the prediction of durability. An overview of the various types of chemical reactions which can occur during corrosion is given by Samson et al. (2000).

The present contribution introduces a model for the corrosion of unprotected concrete subjected to constant pH values between 4.0 and 6.5. At pH values below 4.0 concrete rapidly deteriorates. In this case the concrete requires a protective barrier.

When a concrete surface is subjected to acid attack protons enter the concrete and neutralize the hydroxyl ions contained in the hydration products. This causes mainly calcium, iron, aluminium and sulphate ions to enter the pore solution. These ions then diffuse towards the concrete surface owing to concentration gradients between the pore solution and the attacking acid. The dissolution of the solid hydration products results in an increase in porosity. A corroded layer develops gradually. The speed of penetration of the corrosion front - which is decisive for concrete durability - is determined by (a) the rate of diffusion of the acid through the corroded layer to the reaction front and there (b) the reaction rate of the acid with the concrete.

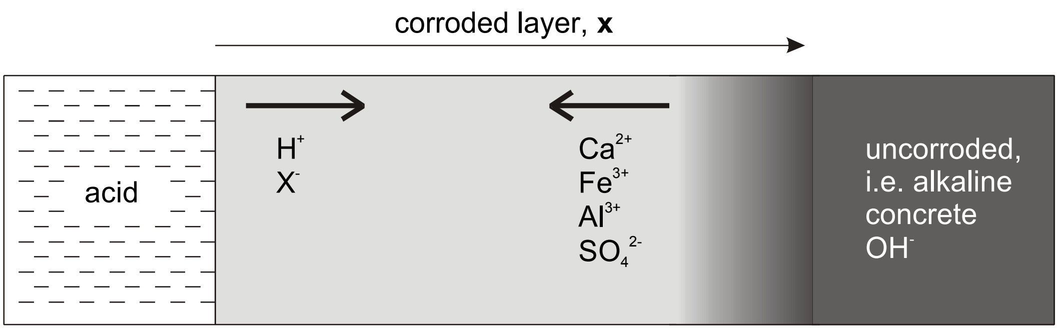


**Figure 1. Corroded layer due to acid attack.**

The corroded layer comprises zones of varying composition and structure which are determined by the distribution of protons within the corroded layer, the different acid resistances of the hydration products and the solubility limits for the dissolved ions. As the pH value within the corroded layer decreases during the course of corrosion, calcium hydroxide (pH 12.5), ettringite (pH 10.7), C-S-H (pH 9) and finally the calcium aluminate and ferrite hydrates decompose successively until a silica gel layer remains at pH values below roughly 2. At pH values between 4.0 and 6.5 phases containing iron and aluminium are still present.

The strength of attack depends on the ability of the acid to dissociate and the solubility of its calcium salt (Revertegat et al. 1992). Although acetic acid dissociates far less readily than the mineral acids it is extremely aggressive owing to the high solubility of its calcium salt. Thus sulphuric acid is less aggressive than acetic acid at similar pH values because soluble gypsum fills surface pores and reduces the ingress of the acid (Pavlík 1994). In the case of sulphuric acid, expansion and destruction of the surface concrete will finally take place.

As opposed to strong mineral acids, weak acids lead to buffering effects which modify the zones described above.

Changes in the corroded layer due to precipitation of salts are also possible. Thus Pavlík (1994) attributed a brown zone, formed adjacent to the uncorroded material during the exposure of hardened Portland cement paste to nitric acid, to the diffusion of $Fe^{3+}$ ions and the subsequent precipitation of ferric hydroxide at pH values above 2. Similarly, the diffusion of $SO_4^{2-}$ in the corroded layer can lead to the precipitation of gypsum.

The corrosion resistance of concrete with quartzitic acid-resistant aggregate depends on the chemical composition of the cement. According to De Belie et al. (1996) the vulnerability of concrete to attack by lactic and acetic acid decreases over the groups: Portland cement without $C_3A$, ordinary Portland cement, Portland cement with fly ash, blast furnace slag cement. In view of the higher corrosion resistance of the calcium aluminate and ferrite hydrates, which decompose at pH values below approximately 4.5 (Biczók 1968), cements with a high $C_3A$ or $C_4AF$ content should improve corrosion resistance. On the contrary, Shi and Stegemann (2000) observed that hardened pastes made with high alumina cement corroded faster in nitric and acetic acids than pastes made with Portland cement.

The corrosion resistance of concrete is affected by the use of mineral additions such as blast furnace slag or fly ash since these materials modify the chemical composition and pore system of the hardened binder as well the composition and permeability of the corroded layer. According to Shi and Stegemann (2000) it is the nature of the hydration products rather than the porosity of the hardened binder that specifies the corrosion resistance.

Fig. 2 shows the effect of binder type and w/b ratio on the deterioration of concrete plates stored for 14 d in a solutions with a pH of 4.5 which contained 0.1 mol/l $Na_2SO_4$ (Dorner, 2000). Compared with samples made from German Portland cement (CEM I) or German blast furnace cement (CEM III), the samples made with high alumina cement (HAC) clearly exhibits the highest resistance against acid attack.

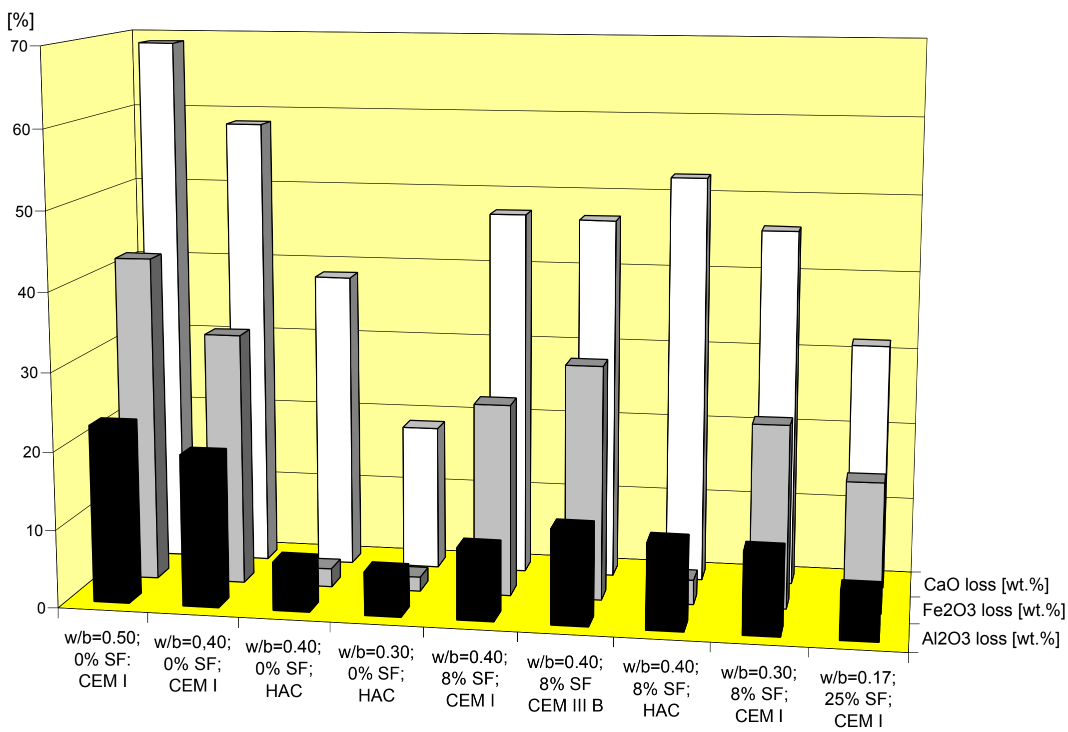


**Figure 2. Loss of Ca, Fe and Al from concrete plates (20×100×150 mm$^3$) stored for 14 d at 50°C in a solution with pH 4.5 containing 0.1 mol/l $Na_2SO_4$ (Dorner 2000)**

## 2 CORROSION OF MORTAR CYLINDERS

Fig. 3 shows the distribution of Ca, Fe and Al within the corroded layer of a mortar cylinder exposed to a buffer solution with a pH of 5.0 for 14 d at 50°C (Dorner 2000). The mortar cylinder was prepared with acid-resistant quartz sand and German Portland cement (CEM I 42,5 R) at a w/c ratio of 0.58. Following storage in the acid, a lathe was used to remove the corroded layer in steps of 0.5 mm down to the undamaged mortar at depth of 6.25 mm. By dissolving the material obtained from each step in hydrochloric acid and chemically analysing the solution composition, it was possible to determine the distributions of Ca, Fe and Al remaining in the corrosion layer. The vertical axis of Fig. 3 gives the remaining amount of Ca, Fe and Al as a percentage of the initial contents of these elements. The initial contents were calculated from the chemical analysis of the cement and the composition of the mortar.

It is apparent that a large proportion of calcium has been removed throughout the thickness of the corroded layer. This is primarily due to the decalcification of C-S-H and calcium hydroxide. Thus the calcium in these phases rapidly dissolved and was able to diffuse through the corroded layer into the surrounding acid. In contrast to this, iron and aluminium were only partially removed from the corroded layer. It therefore appears that the rate of degradation of the hydrate phases containing iron and aluminium is slower. These phases can slow down the corrosion process.

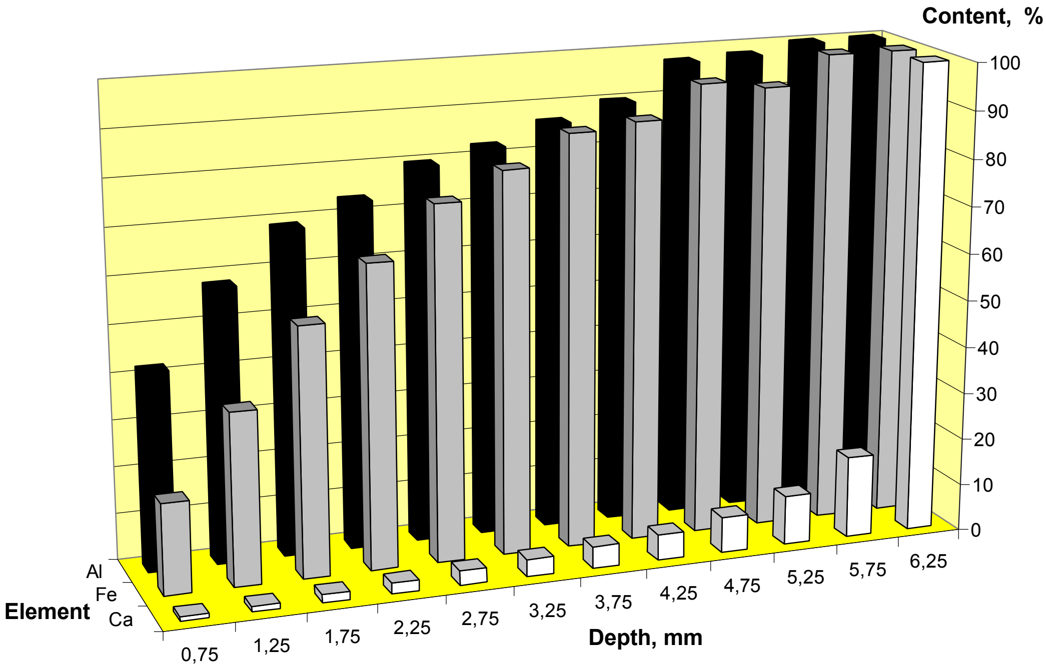


**Figure 3. Distributions of Al, Fe and Ca in the corroded layer of mortar stored for 14 d at pH 5.0 and 50°C (Dorner 2000)**

## 3 SIMULATION OF CORROSION

The exact simulation of the corrosion process requires detailed knowledge, not only of the chemical reactions between the acid and the numerous solid phases, but also of the transport of ions in the pore system. Such a simulation would, ideally, require exact data describing the phases and their amounts as well as a three-dimensional description of the pore system. Schmidt-Döhl and Rostásy (1999), for example, developed a model based on the minimization of the Gibbs free energy of the different solid phases and solution taking part in the chemical reactions leading to corrosion. The model requires knowledge of the chemical or phase composition of the concrete as well as the formation enthalpies of the different phases and the rate constants of the corrosive reactions. Since the present corrosion model should ultimately be applicable to the wide range of concrete compositions used in practice and the different types of acid environments encountered, such a fundamental approach would be extremely difficult. To avoid this problem, integral materials properties (effective diffusion and absorption coefficients, effective rate constants) can be used instead of basic thermodynamic data. This means that the values for the model input parameters should be determined in tests which are as closely linked to the model as possible. The model, the tests and the method of calculating the input parameters from the test results represent a system. Correction factors will, at a later date, be introduced to account for differences between the model and actual experimental results. These factors will be based on physically and chemically definable differences such as effects due to specific surface, additional chemical reactions and interactions between ions in the pore solution.

Currently, the model is being developed by using a number of tests to quantify the corrosion process when cement mortar made with fine acid-resistant quartz sand is stored in an acetic acid / sodium acetate buffer solution. The results of these tests are necessary to (a) confirm the mechanisms and parameters necessary for the simulation and (b) supply the basic input parameters. At a later date, tests will be carried out using strong mineral acids, such as HCl or $HNO_3$. Finally, the application of the model to concrete should only require the results of simple tests, e.g. corrosion depth after a defined storage time at the pH value of the attacking acid.

As already mentioned, the decomposition of the different hydration products depends on the pH of the attacking acid. Thus the rate of a particular corrosion reaction is determined by the concentration of the acid and the type and quantity of the hydration product taking part in the reaction. For example at any given pH, Ca is released from C-S-H and $Ca(OH)_2$ more rapidly than from the ferrite or aluminate hydrates. At first, it will be assumed that the corrosion reactions are of second order type as described by Eqn. (1). The various reactions take place, in principle, simultaneously throughout the depth of the corroded layer.

The dissolution processes result in a corroded layer which is more porous than the original concrete. The porosity is expected to decrease when moving from the surface of the concrete towards the corrosion front where the porosity of the original concrete is reached. This is because the duration of exposure is longest for the concrete surface and zero for the undamaged concrete. Furthermore, the pH of the pore solution increases from the value of the attacking acid at the concrete surface to approximately 12.5 at the corrosion front. This is because proton consumption increasingly dominates proton supply by diffusion. Corrosion experiments indicate that the main increase in pH takes place within a fairly narrow region next to the uncorroded material (Pavlík 1994).

The porosity at an arbitrary distance *x* from the concrete surface within the corroded layer is determined by the combined effect of the different reactions with the local variation of pH over the duration of exposure at *x*. In general, the pH value of the pore solution at *x* decreases with time from about 12.5 to the pH of the attacking acid. The rate of pH decrease is itself a function of the porosity changes within the corroded layer since the protons must permeate the corroded layer to arrive at point *x* and react. In addition, the consumption of protons at *x* by the corrosion reactions affects the local concentration gradient and thus the diffusion of the acid.

### 3.1 Transport and reactions concerning $Ca^{2+}$

The following figure shows schematically the various processes occurring within a volume element of thickness *Dx* at a distance *x* from the concrete surface which lead to changes in the amount of $Ca^{2+}$ in the solid hydration products and the pore solution.

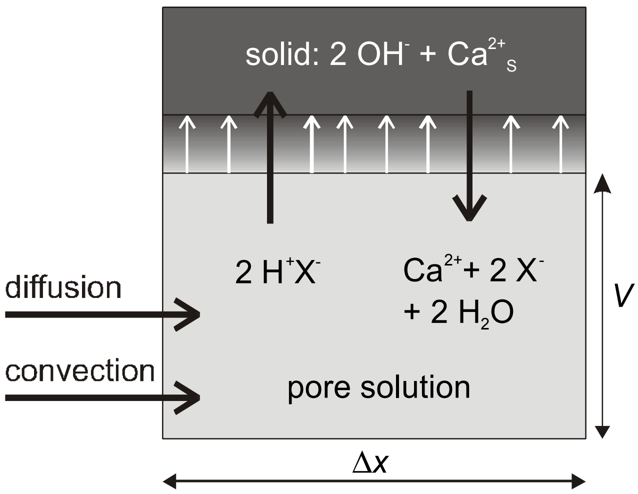


**Figure 4. Processes affecting $Ca^{2+}$ content of hydration products and pore solution in a volume element of thickness *Dx* at a distance *x* from the concrete surface**

The hydroxyl ions in the solid hydration products are neutralized by the protons causing $Ca^{2+}$ ions to enter the pore solution. The rate of neutralization depends on the proton concentration of the pore solution $[H^+]$ (mol/l) and the content of potentially soluble calcium in the solid, $[Ca^{2+}_S]$ (mol/kg). The increase in $[Ca^{2+}]$ also depends on the solution volume *V* and the initial mass of the solid phase $m_0$ participating in the reaction. The latter specifies the total available amount of soluble calcium at the beginning of corrosion. During corrosion, the remaining amount of soluble calcium at *x*, i.e. $[Ca^{2+}_S]\ m_0$ , diminishes. The solution volume *V* is given by the local volume fraction of water-saturated pores into which acid ions are supplied by diffusion and convection. This is essentially the capillary porosity, *P*, if it is assumed that the pores are completely saturated of that diffusion and convection depend on pore size in the same manner.

The reaction kinetics are to a first approximation

$$\frac{\partial [Ca^{2+}]}{\partial t} = K \frac{m_0}{V} [H^+][Ca_S^{2+}] \tag{1}$$

where the variables $[Ca^{2+}]$, $V$, $[H^+]$, and $[Ca^{2+}{}_S]$ are functions of position $x$ and time $t$. The mass $m_0$ is constant. Using the rate constant $K$ it is possible to calculate the increase in dissolved calcium in the pore solution.

The neutralization of the various hydration product phases may be considered to be a reaction between the acid and $Ca(OH)_2$ contained in the phases (see Fig. 4). Consequently, stoichiometry yields the reduction in pH of the pore solution corresponding to Eqn. (1), i.e.

$$\frac{\partial [H^+]}{\partial t} = -2 \frac{\partial [Ca^{2+}]}{\partial t} \tag{2}$$

The reduction in the amount of soluble calcium in the solid phase is given by

$$\frac{\partial [Ca_S^{2+}]}{\partial t} = -\frac{V}{m_0} \frac{\partial [Ca^{2+}]}{\partial t} \tag{3}$$

The supply of acid into $Dx$ and the composition of the pore solution depends on the diffusion flux of the various species. In the simplest form, the diffusion flux $J$ is determined by the concentration gradient of the pore solution and the local water-saturated volume fraction $P$ available for diffusion. Thus

$$J_{H^+} = -D_{H^+} P \frac{\partial [H^+]}{\partial x} \text{ and } J_{Ca^{2+}} = -D_{Ca^{2+}} P \frac{\partial [Ca^{2+}]}{\partial x} \tag{4}$$

The coefficient $D$ is an effective value for the diffusion of ions in the pore solution. It differs from the bulk value for infinitely dilute electrolytes owing to the interactions between ions which decrease the activity coefficients. In addition, the diffusion potential due the different mobilities of the ionic species will tend to slow down the faster ions and speed up the slower ions, see Samson et al. (1999), Tang (1999). The quantity $P$ is also an effective property since only the solution volume effectively contributing to diffusion is considered. It encompasses the effect of pore size distribution, tortuosity and constrictivity of the pore system.

When dry concrete is exposed to an acid solution the ions generated will rapidly enter the concrete by capillary suction. The penetration rate for the front of capillary suction $x_C$ into the concrete and thus the convection of the acid ions is determined by the absorption coefficient $A$ (kg/(m$^2$ s$^{1/2}$) and the capillary porosity P i.e.

$$\frac{\partial x_C(t)}{\partial t} = \frac{1}{2} \left( \frac{A}{P\ r_W} \right)^2 \frac{1}{x_C(t)} \tag{5}$$

Here $r_W$ is the density of water.

### 3.2 Transport and reactions concerning $Fe^{3+}$ and $Al^{3+}$

The transport processes and reactions involving $Fe^{3+}$ and $Al^{3+}$ can be treated analogously to $Ca^{2+}$.

Owing to the neutralization reaction, $Fe^{3+}$ and $Al^{3+}$ ions are accompanied by $Ca^{2+}$ when they are released from the hydrated ferrite and aluminate phases. Thus, as above, a pH reduction is calculated from the increase in the calcium concentration of the pore solution. However, it is likely that different ferric and perhaps aluminium oxide hydrate compounds precipitate or redissolve, thus affecting porosity, diffusion flux and capillary suction. It is also necessary to include this effect during the calculation of the pH of the pore solution.

### 3.3 The acid

Acetic acid has been chosen for the experiments necessary to determine the model input parameters and verify the model. This acid represents the organic acids produced by the decay of organic matter in, for example, waste disposal sites or animal houses (De Belie et al. 1996) and, owing to the high solubility of its calcium salt, attacks concrete with similar aggression to mineral acids.

Acetic acid has the advantage that buffer solutions with sodium acetate can be prepared with pH values ranging from 4.0 to 6.5 which cover the very strong, strong and weak attacks as defined by DIN 4030. Furthermore, acid attack is commonly in the form of large quantities of standing or slowly flowing solution in contact with a concrete surface so that the strength of the attack is not diminished by the reaction with the concrete. This requirement is easily fulfilled by the buffer solution. Another advantage is the high ionic strength of the buffer solution which means that the activity coefficients of ions in the pore solution will not change greatly as ions from the solid material enter the pore solution.

During the simulation of the corrosion process, the properties of the acid must also be taken into account. Dissociation is especially important for weak organic acids and their buffer solutions. The pH of the acid is determined by the equilibrium between the dissociated protons and acid anions with the non-dissociated acid molecules as defined by the dissociation constant of the acid $K_a$, i.e.

$$K_a = \frac{[H^+][X^-]}{[HX]} \tag{6}$$

Differences in the diffusion coefficients of these species and the removal of protons due the reactions with the solid continually disturb the equilibrium according Eqn. (6) thus necessitating the calculation of new solution compositions. The conservation of the number of the components X and H in molecular and ionic form in the solution requires

$$[HX]_1 + [X^-]_1 = [HX]_0 + [X^-]_0 \text{ and } [HX]_1 + [H^+]_1 = [HX]_0 + [H^+]_0 \tag{7}$$

Here 0 denotes the initial unstable composition and 1 the final equilibrium composition.

The solution of Eqns 6 and 7 yield quadratic expressions for the equilibrium composition of the pore solution. It is necessary to calculate the new equilibrium composition of the buffer solution components of the pore solution after each time step $Dt$ of the corrosion model. It is assumed that the equilibrium is not affected by other ions in the solution and that it is achieved instantaneously.

## 4 DETERMINATION OF THE MODEL PARAMETERS

### 4.1 Diffusion coefficients and porosity

The pH value of the pore solution and the time of exposure to the acid vary over the corroded layer. Thus the corrosion model requires values for the diffusion coefficients of the various ions and porosity as a function of the time of exposure to pH values between 4.0 and, theoretically, 12.5.

Thin mortar disks (thickness $l$ = 3 mm , diameter $d$ = 30 mm) are stored for different time periods $t$ up to 100 d in buffer solutions at pH values between 4.0 and nearly 8.5. Following this, the corroded disks are transferred to distilled water so that the ions contained in the pore solution can diffuse out of the disks into the surrounding water. One-dimensional diffusion conditions are obtained by sealing the disk edges with epoxy resin. The concentration of the various ions in the storage water is monitored for up to four days, see Fig. 5. Afterwards the porosity and matrix density of the disks are determined gravimetrically.

In order to determine diffusion coefficients from the experimental data, Fick's second law of diffusion has been solved for the boundary conditions imposed by thin semi-infinite plates to yield an expression for the increase in concentration of an ion in the storage water as a function of the diffusion coefficient. The increase in, for example, the calcium concentration of the water is given by

$$[Ca^{2+}(t)] = \frac{P\,[Ca_0^{2+}]}{\left(P + \frac{4v}{p\,d^2 l}\right)} \left(1 - \sum_{n=1,3,5..}^{\infty} \frac{4}{p^2 n^2} \left(1 - \cos(pn)\right) \exp\left(-\frac{p^2 n^2}{l^2} D_{Ca^{2+}} t\right)\right) \tag{8}$$

Here $[Ca^{2+}{}_0]$ is the initial concentration of calcium in the pore solution and $v$ the storage water volume. Fig. 5 shows examples for the increase in the concentration of ions in the storage water according to Eqn. (8). The curves were calculated for a disk 30 mm in diameter and 3 mm thick immersed for 100 h in 50 ml water. The initial concentration of ions dissolved in the pore solution of the disk was 0.1 mol/l and the porosity of the disk was 30%. The maximum concentration given in the figure is reached when the pore solution and the surrounding storage water have the same concentration.

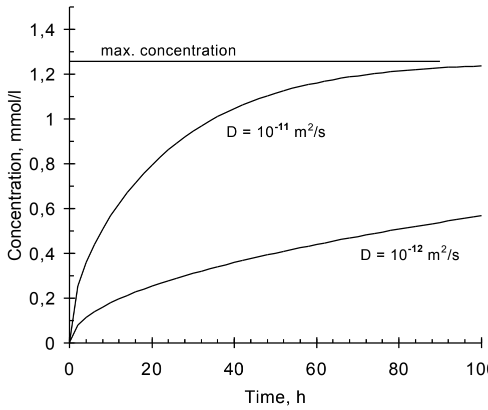


**Figure 5. Diffusion of calcium ions from thin disks into surrounding water**

Values for the diffusion coefficients of the relevant ions, including $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$, are obtained by fitting Eqn. (8) to the appropriate experimental data.

### 4.2 Rate constants for dissolution

The rate constants are determined by solubility measurements with finely ground mortar in order to minimize the effect of diffusion. Samples of initial mass $m_0$ are stored for various lengths of time in buffer solutions of volume $V$ with pH values ranging between 4.0 and 8.5. At the end of the storage period the storage solution is chemically analysed and the dry residue weighed.

The solution of Eqn. 1 for a buffer solution ($[H^+]$ is constant) yields the calcium concentration of the solution as a function of time.

$$[Ca^{2+}] = \frac{m_0}{V}[Ca^{2+}_{S,0}]\left(1 - \exp\left(-K\ [H^+]t\right)\right) \tag{9}$$

Here $[Ca^{2+}_{S,0}]$ is the initial content of potentially soluble calcium in the solid hydration products at the beginning of the acid attack (mol/kg). Values for the rate constants governing the release of $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ and the corresponding initial contents of these ions in the mortar are determined by fitting Eqn. (9) to the experimental data.

Fig. 6 shows results obtained from preliminary solubility measurements. A specimen of finely ground mortar weighing 5 g was placed in 100 ml buffer solution with a pH of 4.5 at 20°C. The concentrations of calcium, iron and aluminium in the buffer solution were recorded a number of times over a 1 h storage period. In the figure, the concentration of calcium, iron and aluminium (symbols) in the solution is plotted against the square root of time. Based on the initial contents, it was estimated that after 1 h more than 95% of the calcium in the sample had entered the solution. Around 95% of the iron and 65% of the aluminium were in the solution.

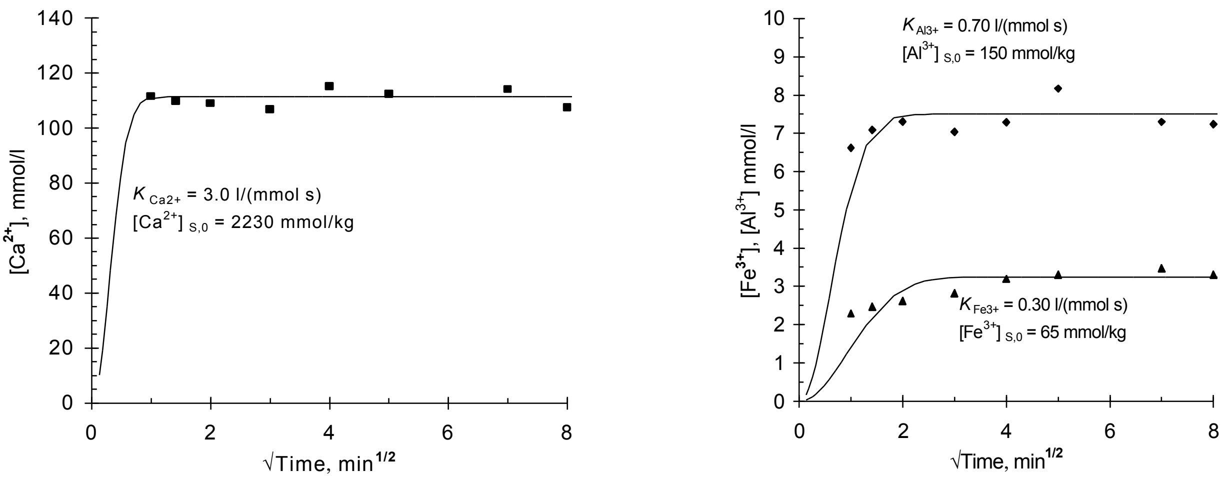


**Figure 6. Increase in concentration of $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ in a buffer solution (pH 4.5) containing finely ground mortar**

In Fig. 6 the experimental results are compared with curves calculated according to Eqn. (9). The values for the rate constant and the content of soluble ions in the mortar are next to the curves. Although the figure illustrates the method by which rate constants can be obtained from the results of solution measurements, a better description of the corrosion reactions is clearly necessary to provide more accurate rate constants.

In principle, the rate constant in Eqn. (9) is valid for the complete range of pH values (4.0 to 12.5) and is therefore an integral value incorporating the effect of many different chemical reactions, precipitation and dissolution processes. A more accurate description of the kinetics requires knowledge of the effect of pH on the various rate constants. In view of the brown zone adjacent to the uncorroded material observed by Pavlík (1994), lower solubility of phases containing iron is expected at higher pH values. This also explains why at pH 4.5 the amount of dissolved aluminium in the ground mortar sample appears to reach a maximum (Fig. 6) well below the total aluminium content, which would completely dissolve at, for example, pH 1.

Eqn. (9) assumes that the rate constant is independent of the ratio of the initial mass of solid to the volume of the liquid in contact with it, $m_0/V$. However, the true ratio within the corroded layer will be much larger than the ratio used in the preliminary solubility measurements. This will affect ionic strength and the solubility limits of the solution. At present experiments are being conducted to ascertain the effect of the ratio $m_0/V$ on the rate constants.

Eqn. (9) also assumes that all the soluble ions contained in the mortar particles are in permanent contact with the acid, i.e. the time needed for the protons to reach any point within the particles is negligible. However, the corrosion reactions will also have some topological character, i.e. $K$ will also depend on specific surface. Since the mortar particles (˜ 60 µm) contain capillary pores, their specific surface is determined by the external and internal areas at which the corrosion reactions occur. This will vary as the dissolution reactions proceed. The change in external specific surface during corrosion is currently being investigated using laser granulometry.

### 4.3 Verification

In order to test the model, mortar cylinders are stored for up to three months in buffer solutions with pH values ranging from 4.0 to 6.5. The calcium, iron and aluminium content of the external buffer solution and the corroded layer, resolved in 0.5 mm steps, are determined by chemical analysis. The thickness of the corroded layer is measured. Finally, the data are compared with the results of the simulation.

## 5 CONCLUDING REMARKS

A model is presented for the prediction of the corrosion of concrete under acid attack at pH values between 4.0 and 6.5 as a function of time. The model considers the diffusion of the acid within the corroded layer and the rate of reaction with the solid phases. Changes in porosity due to corrosion as well as precipitation and dissolution are also considered.

An important part of the model is the simulation of the release of $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ ions into the pore solution and the diffusion of these ions through the corroded layer into the attacking acid. This approach enables the quantification of the corrosion mechanisms based on experimental observations.

The main input parameters of the model are diffusion coefficients for the acid as well as $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$, rate constants for the release of $Ca^{2+}$, $Fe^{3+}$ and $Al^{3+}$ into the pore solution and the associated change in porosity. The parameters are integral, i.e. effective, material properties rather than fundamental thermodynamic quantities, which are chosen in order to simplify the application of the model to practice concretes. The parameters are determined in special experiments tailored to the model.

The values for the diffusion coefficients are determined under instationary conditions by corroding thin fine mortar disks in acetic acid / sodium acetate buffer solutions or other acids, transferring them to distilled water and monitoring the concentration of ions in the water. The porosity of the disks is determined gravimetrically. The rate constants are determined by storing finely ground mortar specimens in buffer solutions and observing the concentration of ions in the solutions. It is necessary to take account of the effect of pH on the corrosion reactions.

The latest experimental results, in particular for high alumina cement, will be presented at the conference.

## 6 ACKNOWLEDGMENT

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# General Methodology Of Test Procedures For Assessment Of Durability And Service Life

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**Summary: A general methodology for assessment of durability and service life by accelerated testing is described. A predictive failure modes and effect analysis is used as a starting point for planning of the accelerated life tests.The method includes several important steps: a) an initial risk analysis of potential failure modes with a checklist of failures and their links with material properties, degradation processes and environmental stress factors, b) screening testing/analysis for service life prediction including pretesting, analysis of material changes resulting from ageing and microclimate characterization, and c) service life prediction from results of accelerated testing with mathematical modelling, life testing and reasonability assessment and validation.**

**The applicability of the proposed methodology is demonstrated by utilizing previous results from a case study on accelerated life testing of selective solar absorber surfaces for domestic hot water production. The work presented forms part of Task 27 "Performance of Solar Facade Components" of the IEA Solar Heating and Cooling Programme.**

**Keywords: Materials, durability, methodology, accelerated tests, service life prediction**

## 1 INTRODUCTION

To achieve successful and sustainable commercialisation, building products must meet three important criteria, namely minimum cost, sufficient performance, and demonstrated durability.

Durability assessment directly addresses all three segments of this triad. First, it permits analysis of life cycle costs by providing estimates of service lifetime, O&M costs, and realistic warranties. Understanding how performance parameters are affected by environmental stresses (for example by failure analysis) allows improved products to be devised. Finally, mitigation of known causes of failure directly results in increased product longevity. Thus, accurate assessment of durability is of paramount importance to assuring the success of solar thermal and building products.

Within the IEA Solar Heating and Cooling Programme, Task 27 on the Performance of Solar Facade Components started at the beginning of year 2000 with the objectives of developing and applying appropriate methods for assessment of durability, reliability and environmental impact of advanced components for solar building facades.

For the work on durability there are two main objectives. The first is to develop a general framework for durability test procedures and service lifetime prediction (SLP) methods that are applicable to a wide variety of advanced optical materials and components used in energy efficient solar thermal and buildings applications. The second is to apply the appropriate durability test tools to specific materials/components to allow prediction of service lifetime and to generate proposals for international standards.

This paper presents a general outline of methodology to meet the first objective. A thorough description of the methodology can be found in a Working Document of IEA Solar Heating and Cooling Task 27 (Carlsson *et al.* 2001)

## 2 GENERAL METHODOLOGY

Many efforts have been made to develop systematic approaches to service life prediction of components, parts of components and materials so that all essential aspects of the problem will be taken into consideration, see e.g. Gaines *et .al.* (1977), Sjöström (1985), CIB W 80/RILEM 71-PSL (1986), ISO 15686-1(2000) and ISO 15686-2 (2001)

In another methodology, which will be focused on in this report, a predictive failure modes and effect analysis serves as the starting point for service life prediction from accelerated life test results as is illustrated schematically in Fig. 1. The analysis is made on the component level.

The diagram in Fig. 1 is based on a similar scheme by Gaines et al. (1977). The diagram in Fig. 1 was originally developed for the purpose of accelerated life testing of selective solar absorber surfaces in a joint case study of Task 10 of the IEA Solar Heating and Cooling Program (Carlsson et al.1994)

- **PENALTY** is the level at which an assessment is made of the economic effects of a component failure. Based on this assumption, it is possible to set a reliability level that must be maintained for a given number of years.
- **FAILURE** is the level at which performance requirements are determined. If the requirements are not fulfilled, the particular component or part of component is regarded as having failed. Performance requirements can be formulated on the basis of optical properties, mechanical strength, aesthetic values or other criteria related to the performance of the component and its materials.
- **DAMAGE** describes the stage of failure analysis at which various types of damage, each capable of resulting in failure, can be identified.
- **CHANGE** is related to the change in the material composition or structure that can give rise to the damage of the type previously identified.
- **EFFECTIVE STRESS** is the level at which various factors in the microclimate, capable of being significant for the durability of the component and its materials, can be identified. An important point here is that it is possible to make quantitative characterisation.
- **LOADS**, finally, is the level that describes the macro-environmental conditions (climatic, chemical, mechanical), and which is therefore a starting point for description of the microclimate or effective stress as above

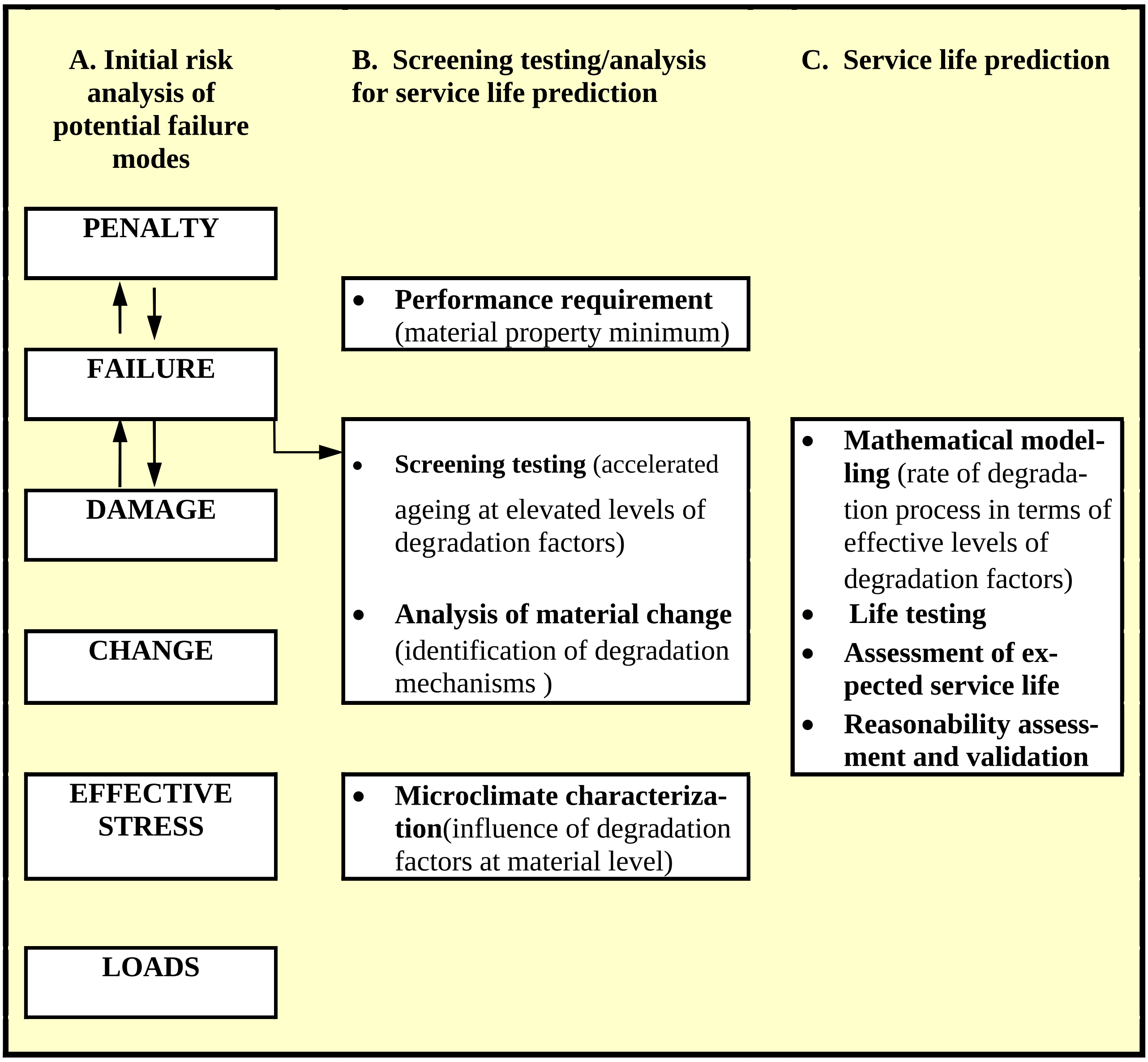



**Figure 1. Failure mode analysis for planning of accelerated tests for service life prediction**

The first phase of work is an initial risk analysis of potential failure modes. Each step in the scheme related to the risk analysis shown on the left hand side of Fig.1 may be related to the subsequent step by an appropriate deterministic or statistical relationship. The second phase of work makes use of the results from the initial risk analysis and involves as major steps screening testing and analysis of associated material changes for identification and confirmation of the most important degradation mechanisms. After the assumptions of the predominating degradation mechanisms have been confirmed the most important degradation factors or environmental stress factors causing material degradation may also be confirmed. Their effective level and duration during service conditions are thereafter assessed by microclimate measurements. The third phase of work involves mathematical modelling, life testing and prediction of service life finely. Reasonability assessment and validation constitute the last steps in the service life prediction scheme.

## 3 INITIAL RISK ANALYSIS OF POTENTIAL FAILURE MODES

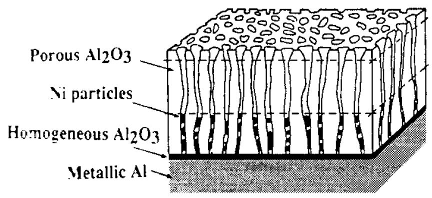
The first step in the scheme illustrated in Fig. 1 is an analysis of potential failure modes with the aim of obtaining

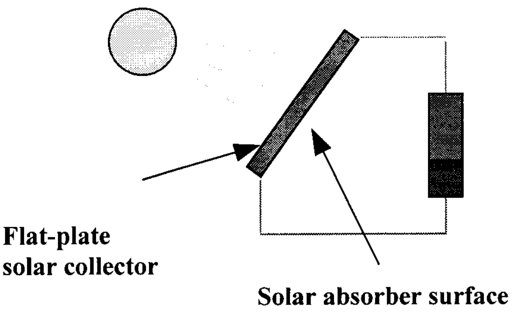
- a checklist of potential failure modes of the component and associated with those risks and critical component and material properties, degradation processes and stress factors,
- a framework for the selection of test methods to verify performance and service life requirements,

**Table 1. Example of result from an initial risk analysis of potential failure modes based on information taken from the IEA Task 10 case study on selective solar absorber surfaces**

**Component/materials:** Selective solar absorber coating of anodised aluminium pigmented with small metallic nickel particles

**Application:** Use in single-glazed flat-plate solar collector for Domestic Hot Water system




**A. Specification of end-user and product requirements on component**

| *Function and general requirements* | *General requirements for long-term performance during design service time* | *In-use conditions and severity of environmental stress* |
|---|---|---|
| -Efficiently convert solar radiation into thermal energy<br><br>-Suppress heat losses in the form of thermal radiation | -Loss in optical performance should not result in reduction of the solar system energy performance (solar fraction) with more than 5%, in relative sense, during a design service time of 25 years | *- Behind glazing in contact with air.*<br>- Casing of collector exchange air with the ambient, meaning that airborne pollutants will enter collector.<br>- If the collector is not rain tight the humidity level of air in the collector may become high<br>- Maximum temperature 200 °C |

**B. Specification of functional properties and requirements on component and its materials**

| *Critical functional properties* | *Test method for deter-mining functional property* | *Requirement for functional capability and long-term performance* |
|---|---|---|
| | | **Functional capability** |
| -Solar absorptance ($\alpha$) | ISO CD 12592.2 | $\alpha > 0.92$ |
| -Thermal emittance ($\varepsilon$) | ISO CD 12592.2 | $\varepsilon < 0.15$ |
| -Adhesion (ad) | ISO 4624 | ad >0.5 MPa |
| | | **Long-term performance** |
| | | $PC = - \Delta\alpha + 0.25\ \Delta\varepsilon \leq 0.05$ |

- a framework for describing previous test results for a specific component and its materials or a similar component and materials used in the component and classifying their relevance to the actual application, and
- a framework for compiling and integrating all data on available component and material properties and material degradation technology.

From a practical point of view, but also from an economic viewpoint, an assessment of durability or service life has to be limited in its scope and focused on the most critical failure modes. An important part of the initial step in such an assessment is therefore estimating the risk associated with each of the potential failure modes of the component.

The programme of work in the initial step of service life assessment may be structured into the following activities (Carlsson, 1993):

a) Specify from an end-user point of view the expected function of the component and its materials, its performance and its service life requirement, and the intended in-use environments;

b) Identify important functional properties defining the performance of the component and its materials, relevant test methods and requirements for qualification of the component with respect to performance;

c) Identify potential failure modes and degradation mechanisms, relevant durability or life tests and requirements for qualification of the component and its materials as regards durability. When identifying potential failure modes, it is important to distinguish between 1) failures initiated by the short-term influence of environmental stress, the latter representing events of high environmental loads on the component and its materials, 2) failures initiated by the long-term influence of environmental stress, the latter causing material degradation so that the performance and sometimes also the environmental resistance of the component and its materials gradually decrease.

d) Estimation of risks associated with different failure modes

e) The result of the initial risk analysis of potential failure modes may be documented as shown in Table 1 and Table 2 using information from a case on accelerated life testing performed in Task 10 of the IEA Solar Heating and Cooling Programme (Carlsson *et al.* 1994)

The first activity specifies in general terms the function of the component and service life requirement from an end-user and product point of view, and from that identifies the most important functional properties of the component and its materials, see Table 1.

How important the function of the component is from an end-user and product point of view needs to be taken into consideration when formulating the performance requirements in terms of those functional properties. If the performance requirements are not fulfilled, the particular component is regarded as having failed. Performance requirements can be formulated on the basis of optical properties, mechanical strength, aesthetic values or other criteria related to the performance of the component and its materials. Defining performance requirement should be accompanied by an assessment of the economic effects of a component failure. Based on this, it is possible to define a service life requirement or set a reliability level that must be maintained for a given number of years.

**Table 2. Risk assessment of potential failure modes by use of FMEA applied to the selective absorber surface studied in IEA Task 10, see Table 1**

| *Failure mode / Degradation process* | *Severity (S) (rating number)* | *Probability of occurrence($P_O$) (rating number)* | *Probability of discovery($P_D$) (rating number)* | *Rating-number for risk ($RPN = S \times P_O \times P_D$)* |
|---|---|---|---|---|
| | | | | |

| Severity | Rating number |
|---|---|
| No effect on product | 1 |
| Minor effect on product but no effect on product function | 2-3 |
| Risk of failure in product function | 4-6 |
| Certain failure in product functioning | 7-9 |
| Failure which may affect personal safety | 10 |

| Probability of detection | Rating number |
|---|---|
| Failure which always is noted. Probability for detection > 99.99% | 1 |
| Normal probability of detection 99.7% | 2-4 |
| Certain probability of detection >95% | 5-7 |
| Low probability of detection >90% | 8-9 |
| Failures will not be found - cannot be tested | 10 |

| Probability of occurance | Rating number |
|---|---|
| Unlikely that failure will occur | 1 |
| Very low probability for failure to occur | 2-3 |
| Low probability for failure | 4-5 |
| Moderate probability for failure to occur | 6-7 |
| High probability for failure to occur | 8-9 |
| Very high probability for failure to occur | 10 |

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**C. Service reliability/service life**

| *Failure/Damage mode / Degradation process* | *Degradation indicator* | *Critical factors of environmental stress/ Degradation factors and severity* | *Estimated risk of failure/damage mode from FMEA* | | | |
|---|---|---|---|---|---|---|
| **Unacceptable loss in optical performance** | PC = - $\Delta\alpha + 0.25\ \Delta\varepsilon$; Adhesion | | *S* | $P_O$ | $P_D$ | Risk *RPN* |
| (A) High temperature oxidation of metallic nickel | Reflection spectrum Vis-IR | High temperature | 7 | 2 | 8 | 112 |
| (B) Electrochemical corrosion of metallic nickel | Reflection spectrum Vis-IR | High humidity, sulphur dioxide (atmospheric corrosivity) | 7 | 5 | 5[1] | 175 |
| (C) Hydatization of aluminium oxide | Reflection spectrum IR | Condensed water, temperature | 7 | 7 | 4[1] | 196 |
| | | [1] Result of glazing failure | | | | |

Potential failure modes and important degradation processes should be identified after failures have been defined in terms of minimum performance levels. In general, there exist many kinds of failure modes for a particular component and even the different parts of the component and the different damage mechanisms, which may lead to the same kind of failure and may sometimes be quite numerous.

The objective of analysis is to identify potential failure/damage modes and mechanisms that may lead to material degradation and the development of damage, and associated critical factors of environmental stress or degradation factors, see the example in Table 2.

The risk or risk number associated with each potential failure/damage mode identified can be estimated by use of the methodology of FMEA (Failure Modes and Effect Analysis) in a simplified way, see IEC Standard (1985) for a review of the FMEA methodology. The estimated risk number is taken as the point of departure to judge whether a particular failure mode needs to be further evaluated or not. The estimated risk number may also be used to determine what kind of testing is needed for qualification of a particular component and its materials, see the example in Table 2.

## 4 SCREENING TESTING/ANALYSIS FOR SERVICE LIFE PREDICTION

### 4.1 Screening testing by accelerated ageing

Screening testing is thereafter conducted with the purpose of qualitatively assessing the importance of the different degradation mechanisms and degradation factors identified in the initial risk analysis of potential life-limiting processes.

When selecting the most suitable test methods for screening testing, it is important to select those with test conditions representing the most critical combination of degradation factors, see example in Table 3.

**Table 3. Programme for screening testing in the IEA case study on solar absorber absorbers**

| *Possible degradation mechanism* | *Critical periods of high environmental stress* | *Suitable accelerated test methods and range of degradation factors* |
|---|---|---|
| (A) High temperature oxidation of metallic Ni particles | Stagnation conditions of solar collector at high levels of solar irradiation (no withdrawal of heat from the collector) | Constant load high temperature exposure tests in the range of 200-500 °C |
| (B) Electrochemical corrosion of metallic Ni particles at high humidity levels and in the presence of sulphur dioxide | Under starting-up and under non-operating conditions of the solar collector when the outdoor humidity level is high | Exposure tests at constant high air humidity (75 – 95 % RH), constant temperature (20- 50°C), and in the presence of sulphur dioxide (0-1 ppm) |
| (C) Hydratization of aluminium oxide and electrochemical corrosion of metallic Ni particles by the action of condensed water | Under humidity conditions involving condensation of water on the absorber surface | Exposure tests under constant condensation (sample surface cooled 5 °C below surrounding air which is kept at 95 % RH) and temperature conditions ranging from 10 - 90 °C |

### 4.2 Analysis of material change during ageing

Using artificially aged samples from the screening testing, changes in the key functional properties or the selected degradation indicators are analysed with respect to associated material changes. This is made in order to identify the predominant degradation mechanisms of the materials in the component. When the predominant degradation mechanisms have been identified also the predominant degradation factors and the critical service conditions determining the service life will be known.

Screening testing and analysis of material change associated with deterioration in performance during ageing should therefore be performed in parallel. Suitable techniques for analysis of material changes due to ageing may vary considerably. In Table 4 an example from the IEA absorber surface case study is shown that demonstrates how different techniques for analysing material changes resulting from ageing can be used to get information on what material degradation mechanisms are contributing to deterioration in performance.

**Table 4. Techniques that were used in the IEA Task 10 solar absorber case study for analysis of material change upon durability testing**

| *Degradation mechanism* | *Techniques for analysis of material changes* | *Results* |
|---|---|---|
| (A) High temperature oxidation of metallic Ni particles | - UV-VIS-NIR reflectance spectroscopy<br>- AES depth profiling<br>- SEM-EDX<br>- XRD | - Reduction of absorption in solar range corresponding to reduction in metal concentration<br>- Formation of Ni oxides<br>- Small changes in surface morphology<br>- Formation of NiO |
| (B) Electrochemical corrosion of metallic Ni particles at high humidity levels and in the presence of sulphur dioxide | - UV-VIS-NIR reflectance spectroscopy<br>- FTIR-IR reflectance spectroscopy<br>- AES depth profiling<br>-SEM-EDX | - Reduction of absorption in solar range corresponding to reduction in metal concentration<br>- Formation of sulphate<br>-Increase in surface concentration of Ni accompanied with sulphur at the surface<br>- Surface morphology affected and detection of sulphur |
| (C) Hydratization of aluminium oxides and electrochemical corrosion of the metallic particles by the action of condensed water | - UV-VIS-NIR reflectance spectroscopy<br>- FTIR-IR reflectance spectroscopy<br>- AES depth profiling<br>- SEM-EDX | - Some changes hard to explain<br>- Formation of hydrated forms of alu-minium oxide leading to increased thermal emittance<br>- Change in surface structure<br>- Considerable change in surface morphology |

### 4.3 Microclimate characterisation for service life prediction

In order to be able to predict expected service life of the component and its materials from the results of accelerated ageing tests, the degradation factors under service conditions need to be assessed by measurements. In Table 5 measurement techniques that were used in the IEA Task 10 absorber case study previously reviewed are given as an example of what factors were needed to take into consideration in this study. It is of extreme importance to characterize the service conditions in terms relevant for the most important degradation mechanisms identified for the materials of the component but also in terms relevant for and convertible into the test conditions for the environmental resistance tests to be used for accelerated life testing.

#### 4.3.1 Distribution functions of single degradation factors or combinations of degradation factors

If only the dose of a particular environmental stress is important then the distribution or frequency function of a degradation factor is of interest. In the IEA absorber case study, only the distribution in the absorber temperature during service conditions was needed for predicting the service life limited by high temperature degradation, see Fig.1, left diagram. In case of service life prediction considering degradation caused by the action of high humidity and condensed water, both air humidity and surface temperature were taken into account, see Fig. 1, right diagram.

**Table 5 Techniques that were used in the IEA Task 10 absorber case study for measurement of degradation factors in solar collectors operating under service conditions**

| *Degradation mechanism* | *Degradation factors/*<br>*Measurement variables* | *Sensors* |
|---|---|---|
| (A)<br>High temperature oxidation of metallic Ni particles | *Temperature:* Surface temperature of absorber plate | Pt sensors in holders screwed directly on the absorber plate. To accomplish a good thermal contact heat conducting compound was used. |
| (B)<br>Electrochemical corrosion of metallic Ni particles at high humidity levels and in the presence of sulphur dioxide | *Atmospheric corrosivity*: Measurement of corrosion mass loss rate of standard metal specimens<br>*Air pollutants*: Measurement of sulphur dioxide concentration inside and outside of the solar collector. | Metal coupons of carbon steel, zinc and copper and evaluation of corrosion mass loss according to ISO 9226<br>Exposed metal coupons analysed in respect of the sulphate content of the corrosion products by EDX<br>UV-fluorescence instrument for direct measurement of sulphur dioxide concentration in the air outside and inside of the solar collector |
| (C)<br>Hydratization of aluminium oxide and electrochemical corrosion of metallic Ni particles by the action of condensed water | *Humidity:* Measurement of air humidity in the air gap between the absorber plate and glazing cover of the collector<br>*Time of condensation:* Measurement of specular reflectance of absorber surface<br>*Surface humidity:* Measured relative air humidity converted to relative humidity on surface by use of measured surface temperatures | Capacitance humidity sensors carefully shielded from solar radiation and thermal radiation of the ambient.<br>Special designed reflectance mode condensation sensor |

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## 5 SERVICE LIFE PREDICTION FROM RESULTS OF ACCELERATED TESTING

### 5.1 Mathematical modelling

To perform an accelerated test, D, means that the level of at least one stress factor, X, causing degradation is kept at a higher level relative to the situation in service. Consequently this means the time to failure in the accelerated test, $\tau_{f,D}$ , will be shorter relative to service life, $\tau_s$. The ratio between the latter and the former is commonly referred to as the acceleration factor, A. If the applied stress is constant in time also for service conditions, the acceleration factor A can be expressed as

$$A = \tau_S / \tau_{f,D} = a_X = g(\underline{X}_D) / g(X_S) \quad (1)$$

where the expression $g(\underline{X}_D) / g(\underline{X}_s)$ is called the time transformation function or the acceleration factor function. Examples of time transformation functions can be found in e.g. reports by Martin (1982), Carlsson et al. (1988 and 2001). The time transformation functions, $a_X$ , used in the absorber case study are shown in Table 6.

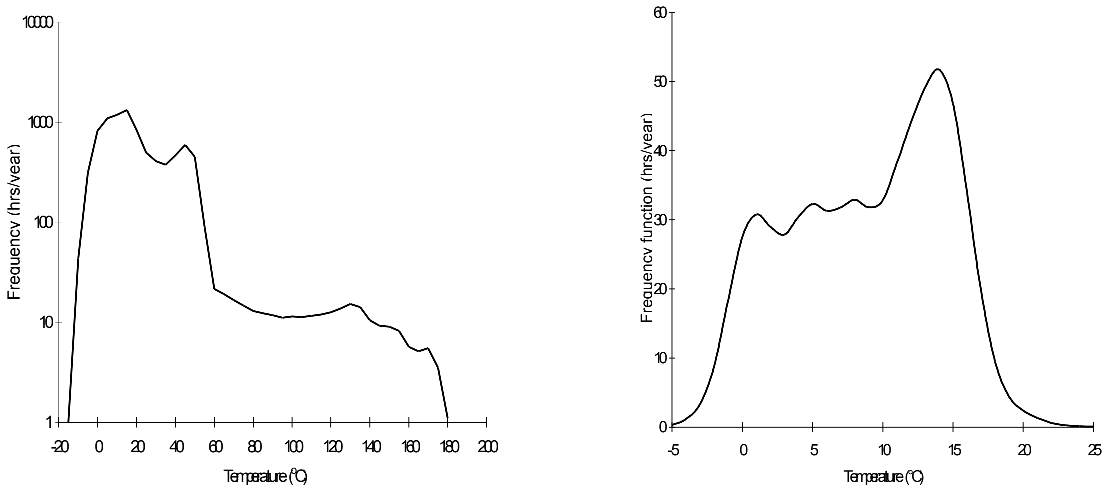


**Figure 1 Results from measurement of microclimate for the absorber in the IEA absorber surface case study - Left diagram: Absorber temperature frequency function for one year. For one month of the year the collector was under stagnation conditions - Right diagram: Absorber temperature frequency function when RH ≥ 99% of that year. Metallic mass loss due to corrosion of zinc was determined to 0.3 g/m$^2$, year**

### 5.2 Accelerated life testing and assessment of expected service life

Accelerated life testing means to quantitatively assess the sensitivity to the various degradation factors on the overall deterioration of the performance of the component and its materials in terms of the mathematical models set up to characterize the different degradation mechanisms identified. Life testing therefore requires conducting a series of tests.

From the accelerated life test results the parameters of the assumed model for degradation are determined and the service life then estimated by extrapolation to service conditions. If the service conditions vary, effective mean values of stress need to be assessed from measured service stress data (Carlsson *e al.* 2001), see also Fig 1.

As a result, it may be possible to express the importance of different degradation mechanisms in terms of expected service life values, see example from the absorber surface case study in Table 6.

### 5.3 Reasonability assessment and validation

By use of accelerated life testing, potential degradation mechanisms limiting the service life of a component may be identified. However, it is important to point out that it is only the service life determined by the material degradation mechanisms observed in the accelerated tests at relative high levels of stress that can be assessed. Life-limiting degradation mechanisms may exist that cannot be identified by way of accelerated life testing because the knowledge and experience in what may cause degradation of a particular material in a component may be too limited.

**Table 6. Estimated service life of the nickel- pigmented anodised aluminium absorber surface in the IEA case study. Values are given for the different degradation mechanisms and assuming that the different degradation mechanisms are acting alone**

| *Degradation mechanism* | *Time transformation function ($a_X$)* | *Estimated service life with PC = -?a + ?e < 0.05* [1] *(years)* |
|---|---|---|
| (A)<br>High temperature oxidation of metallic Ni particles | $a_T = \exp[-(E_a / R)\cdot(1/T_D - 1/T_s)]$<br>$E_a$ = activation energy<br>R = general gas law constant<br>$T_D$ = temperature of test<br>$T_s$ = effective mean temperature at service | $>10^5$ |
| (B)<br>Electrochemical corrosion of metallic Ni particles at high humidity levels and in the presence of sulphur dioxide | $a_{Co} = \tau_{M,s} / \tau_{M,D}$<br>$\tau_{M,s}$ = time to reach a certain extent of corrosion of reference metal in service<br>$\tau_{M,D}$ = time needed to reach the same extent of corrosion of reference metal in test D<br>(zinc used as reference metal) | 12<br>(The coating is assumed to be installed in a non-airtight highly ventilated collector)<br>34<br>(The coating is assumed to be installed in an airtight collector with controlled ventilation ) |
| (C)<br>Hydratization of aluminium oxide and electrochemical corrosion of metallic Ni particles by the action of condensed water | $a^{-1}_{T,H} = \tau_H \cdot \exp(-\frac{E_{H,T}}{R}(T^{-1}_{H,eff} - T^{-1}_D))$<br>$T_{H,eff}$ = effective mean temperature of the absorber surface when the relative humidity in the air gap is equal to or higher than 99 %.<br>$\tau_H$ = the time fraction of the year, time-of-wetness, during which the relative humidity in the air gap is equal to or higher than 99 %.<br>$E_{H,T}$ = Arrhenius activation energy | 9<br>(The coating is assumed to be installed in an non- airtight highly ventilated collector) |

[1] PC = 0.05 corresponds to a decrease in the solar system performance of 5%

The best approach in validating an estimated service life from accelerated testing, therefore, is to use the results from the accelerated life tests to predict expected change in material properties or component performance versus service time and then by long-term service tests check whether the predicted change in performance with time is actually observed or not.

The results of validation tests therefore can be used to revise a predicted service life and form the starting point also for improving the component tested with respect to environmental resistance, if so required. It should be remembered that the main objective of accelerated life testing is to try to identify those failures, which may lead to an unacceptable short service life of a component. In terms of service life, the main question is most often, whether it is likely or not, that the service life is above a certain critical value.

In order to validate the predicted service life data from accelerated life testing in the IEA absorber case study the actual service degradation in optical performance of the nickel pigmented anodized aluminium absorber coating was investigated (Carlsson *et al.* 2000). Samples from the coating taken from collectors used in solar DHW systems for time periods of ten years or more were analysed for that purpose. It could be concluded that the agreement between degradation data determined for the absorber samples from the DHW systems and that from accelerated life testing from the Task 10 study was astonishingly good both from a quantitative and a qualitative point of view. For the absorber coating in a properly designed solar collector, the service life seems good enough. For the absorber coating in a not air tight solar collector, probably because of glazing failures, the humidity level is raised to such high levels that the service life is reduced to an unacceptable level.

## 6 ACKNOWLEDGEMENT

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# Performances Lead
# The Way To Service Life Prediction

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**Summary: Service life planning and service life prediction is today ruled by the ISO 15686 standard. This standard can guide user to find out the Reference Service Life (RSL) of components and materials from both laboratory aging tests and field exposures. In particular, during last years researches on durability of building materials gave considerable results on building materials behavior during their service life, but there is still a lot of work to be done on methods for predicting the Estimated Service Life (ESL) of components or materials in real projects.**

**This paper presents results from a research developed at the "Dipartimento di Ingegneria dei Sistemi Edilizi e Territoriali – Polytechnic of Milan" on the durability of building components, and shows a method, based on a performance approach, for estimating the service life of a component starting from experimental data.**

**The transition from experimental data to ESL can be done, according to the ISO 15686, in three different ways: a) factor method, b) engineering methods and c) stochastic methods. The tool developed at the DISET can be compared to an engineering method; it allows estimating the service life of a component by the definition of limits to the degradation of its performance characteristics.**

**Starting from user requirements on indoor climate, in particular on thermal comfort and humidity control, and from experimental data on the behavior over time of building components, this method allows the user to define limits for performance characteristics of building components. These are boundaries that are not to be passed during the degradation of building components if we want the indoor climate to satisfy user requirements and therefore they determine the service life of the component. We call these limits "performance limits".**

**Results from a test application on a standard Italian bedroom are shown in order to explain how the tool can be used to predict the service life of a traditional Italian masonry. In particular, performance limits are computed starting from the following required performance: control of surface condensation, control of interstitial condensation, thermal insulation, and thermal inertia in winter conditions.**

**A part of this paper is aimed to show sensitive analysis and risk analysis done on results obtained. A sensitive analysis allows us to highlight most influencing characteristics, i.e. parameters that affect the indoor climate the most when their values change. Risk analysis gives us information on the influence of errors during different stages of the building process on indoor climate.**

**Keywords. Service Life Prediction, Engineering Methods, Building Components Performances**

## 1 INTRODUCTION

Studies on durability of buildings and building components don't date too match back in time, main passages in this field are:

- 1997: "Prediction of service life of building materials and components" by CIB W80 – RILEM 71 PSL (nowadays RILEM 175 SLM);

- 1992: “Guide to durability of buildings and building elements, products and components” by British Standards Institution;
- 1993: English translation of “Principal guide for service life planning of buildings” by Architectural Institute of Japan;
- 1999: ¨ISO 15686 “Service life planning”, currently the *state of the art* in this field.

ISO standard is divided into five main parts:

1. general principles for service life prediction of buildings and building components;
2. performance tests and reference service life;
3. service life audit;
4. performance data;
5. planned and reactive maintenance.

In particular, part two describe a test method to predict the reference service life of building components, i.e. the period of time, after installation, during which a building component meets or exceeds performance requirements in normal conditions of use and maintenance. At the DISET – Polytechnic of Milan we are currently working, together with some researcher of the LTS - SUPSI, on an experimental program to evaluate Reference Service Life of a typical Italian masonry (first results in Maggi *et al.* 1999). The test procedure used is very similar to the one described in part 2 of ISO 15686, and the results we are obtaining are to be considered as the RSL of the different kind of masonries tested.

The problem we faced, once first results were obtained, was how to obtain the Estimated Service Life from the reference service life values we got. The bridge between RSL and ESL is the object of researches all over the world, members of CIB W80 - RILEM 175 SLM suggested that each method should be classified in one of the following categories:

- factorial method;
- probabilistic methods;
- engineering methods.

### 1.1 Factorial method

The factorial method is the simplest possible approach to service life prediction and consists of combining RSL value with seven different factors, according to the following equation:

$$ESL = RSL \cdot A \cdot B \cdot C \cdot D \cdot E \cdot F \cdot G$$

Each factor measures the effects of the difference between the standard conditions of use and maintenance according to which RSL has been obtained and the actual in use conditions of the components. The meaning of each factor, according to ISO 15686-1, is:

A. quality of component;
B. design level;
C. work execution level;
D. indoor environment;
E. outdoor environment;
F. in use conditions;
G. maintenance levels.

Even if the factor method is easy and quite inexpensive to be used in real project, it seems to hold the actual complexity of degradation processes of small account.

### 1.2 Probabilistic methods

In these methods degradation is generally regarded as a stochastic process, for each property during each time period, a probability of deterioration is defined (models like the Markov chain model are usually used). These methods require fairly sophisticated inputs in the form of probabilities, which are not easily estimated, and require quite some effort to be put in. Examples of use of stochastic method in research projects and large engineering projects can be found in literature, see Abraham (1999), Leira (1999), Lounis (1998), but most of them are focused on one single material (almost always on reinforced concrete) and on one single aging agent (quite often chloride attack). The efforts needed to use these kinds of methods in real projects made probabilistic method economically usable only in large projects.

### 1.3 Engineering methods

Once characteristics of factor method and probabilistic methods where clear to everybody, researchers start feeling the need for a compromise solution, methods which are as easy to use as the factor method and describe actual degradation processes like probabilistic methods do. These methods were called engineering (design) methods.

Engineering methods must have the same degree of complexity of other common design task (i.e. structural analysis, thermal analysis, etc.) and they may have probabilistic inputs but these inputs must be linked by simple, determinate equations.

No examples of engineering methods could be found in literature when we obtained first experimental RSL for tested component and we decided to try to develop our own methods.

## 2 PERFORMANCE LIMITS

The starting point was the definition of service life found in ISO 15686: “*period of time after installation during which a building or its parts meet or exceed the performance requirements* ” and the main aim was to tie the estimated service life to igro-thermal performances of buildings.

A brief description of the performance limits method is the following:

- first of all it is necessary to define performance requirements for each part of the building.
- We thought important to be tied to igro-thermal performances because these are directly felt by users. On the other hand this choice bind estimated service life values to hypothesis made on indoor space during the design stage, because some of the computational models used made assumption on rooms dimensions. We used minimum performance requirements stated by Italian laws in the case study;
- then performance requirements for each building component must be computed.
- The translation of performance requirements of spaces of the building to performance requirements of building components is quite difficult. Computational model to verify indoor climate parameters lack and are often complex, when possible we used algorithms stated in international standards. During this steep minimum values stated by Italian laws for (igro-thermal) performance of spaces were translated into minimum values for building components performances.
- Next step is to assign performance tasks to each layer of the component. Each minimum value for building components performances generates one or more boundary values for characteristics of materials of each layer of building components. If these boundary values are crossed the component will be no longer able to satisfy user needs. We call these boundary values for the characteristics of each layer of the component *performance limits.*
- the comparison between the performance limits and the degradation of building materials characteristics over time gives the estimated service life.

### 2.1 Case study

As a test application performance limits for two different kind of building envelope components were computed. Example of results obtained for an envelope component (‘Fig. 1’) made of internal hollow brick, thermal insulating layer and external perforated bricks coated on both side with plaster and finished with water paint, are presented.

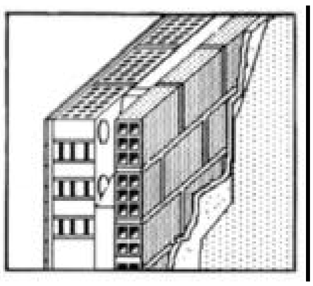

**Figure 1 – Section of the envelope component**

Starting from the study of how the envelope of buildings works, five major contributions on igro-thermal performances of buildings have been highlighted. We called these five contributions “*essential requirements*” of building envelope components. The following Table 1 shows most important requirements to building envelope components and characteristics of materials that influence their behavior in relation to indoor climate.

**Table 1. Requirements and performance characteristics of building envelope components**

| *Essential requirements* | *Performance characteristics* |
|---|---|
| Control of surface condensation | Thermal conductivity |
| Control of interstitial condensation | Thermal conductivity, water vapor permeability |
| Control of Thermal insulation | Thermal conductivity |
| Control of Thermal inertia in winter conditions | Thermal conductivity, specific heat, density |
| Control of Thermal inertia in summer conditions | Thermal conductivity, specific heat, density |

Performance associated to thermal insulation and thermal inertia can be evaluated only if hypotheses on a room are made because these performances do not depend only on building components, they are related to a building space. We use a standard Italian single bedroom of nine square meters (standard minimum) and three meters high. The room has a window in the center of one of its four walls of 1.125 square meters (standard minimum) with a 1.10 meters high railing (standard minimum).

First of all a sensitivity analysis of the model was made in order to gain information on how every parameters of the model influences the results. The value of each input characteristic (material, climate and space) was changed of +50% and performances related to essential requirements were computed. Then the same calculation was made with a –50% changing and the difference between the two values of each performance was computed, the bigger this difference is the more the characteristic influence performance limits.

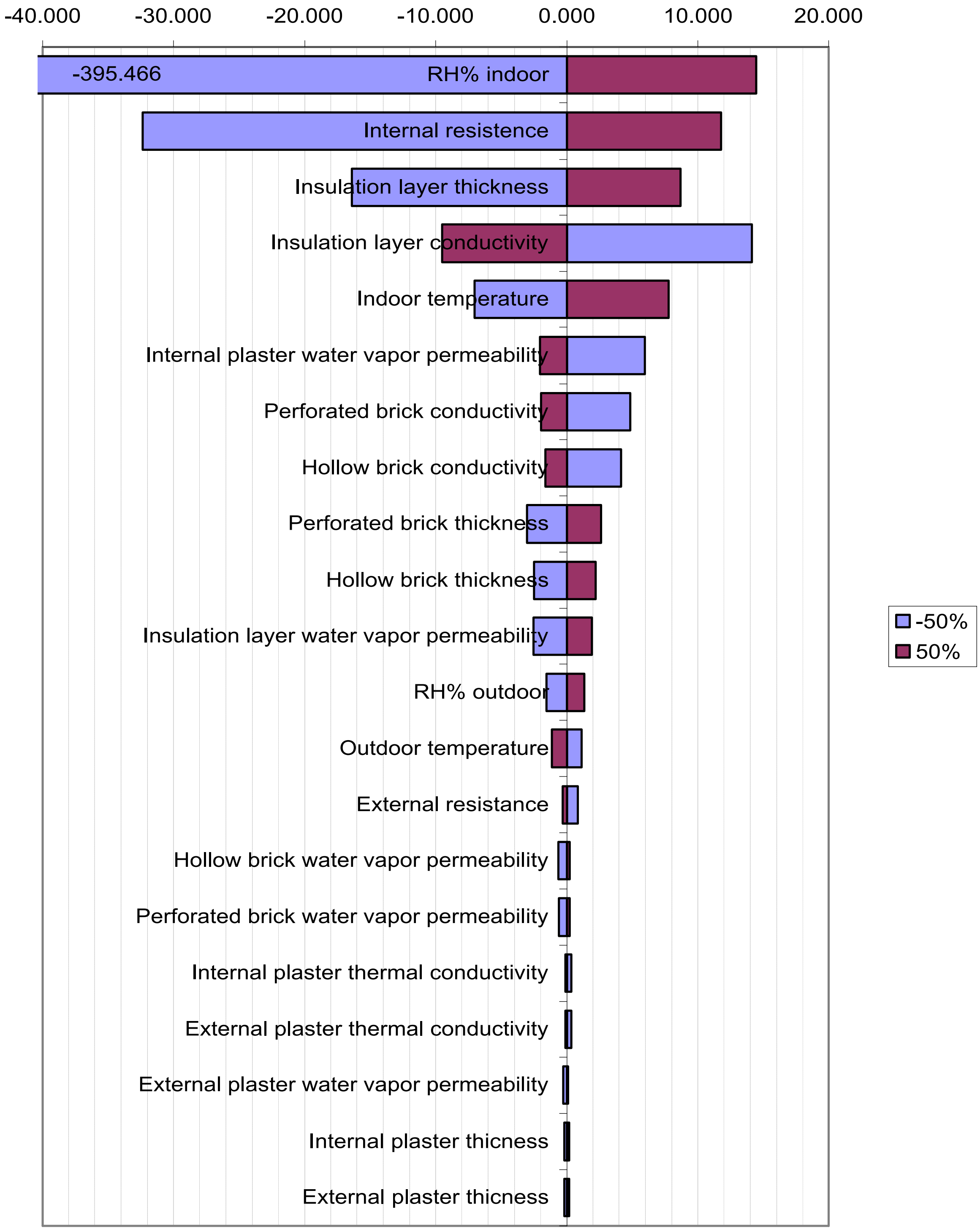


**Figure 2 – Control of surface condensation: results of sensitivity analysis**

An example of results obtained from sensitivity analysis is shown in ‘Fig. 2’, it shows material characteristics and hypotheses on climate influencing surface condensation for the external wall used for the test application. Characteristics are ranked from the most influencing (top) to the less influencing (bottom).

Results coming out from sensitivity analysis were summarized in a table, shown in ‘Fig. 3’,where the influence of every performance characteristics of the component on the computational model adopted is highlighted with different colour, from red (more influencing) to green (less influencing).

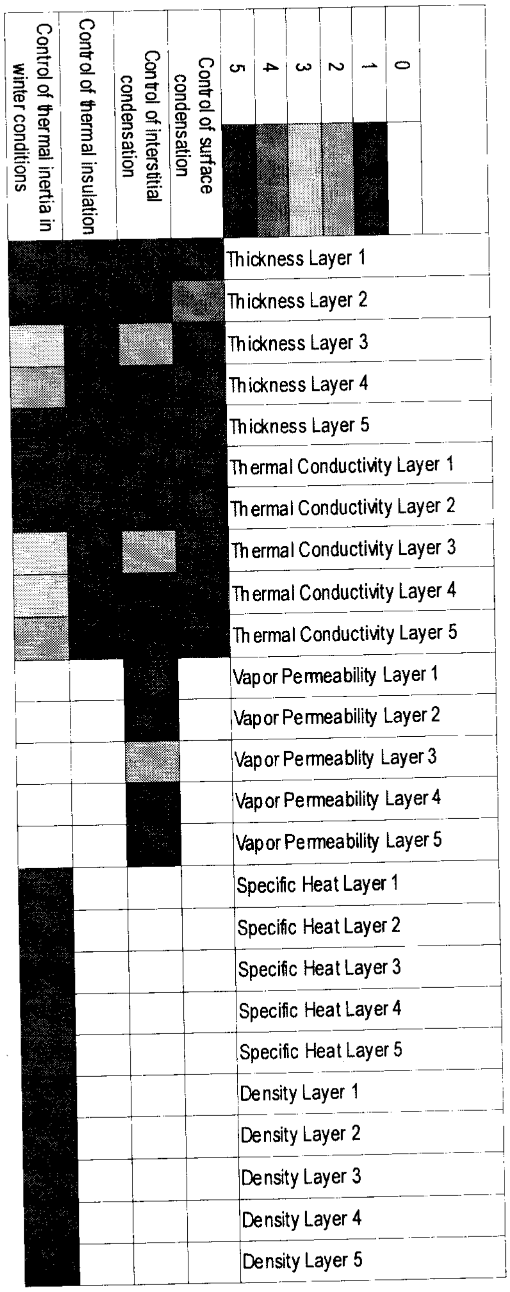


**Figure 3 –Sensitivity table for performance characteristics**

When sensitivity analysis was finished and the computational model was well known and tested performance limits for every performance characteristics have been estimated. ‘Figure 4’ shows the performance limit for thermal conductivity and water vapor permeability of the insulation layer computed for the requirement of control of interstitial condensation.

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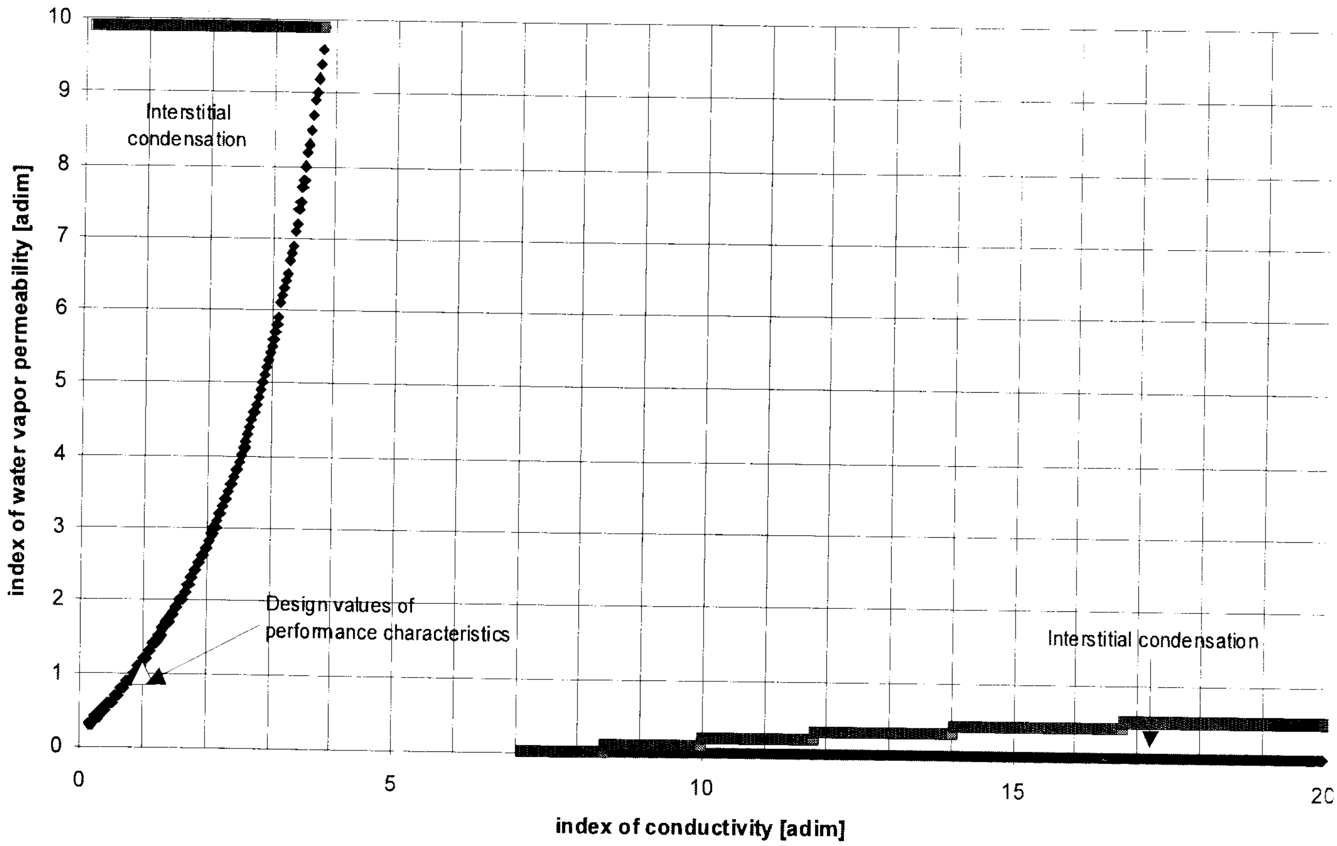


**Figure 4 – Performance limit for the insulation layer (control of interstitial condensation)**

The figure shows two regions of values of thermal conductivity and water vapor resistance causing interstitial condensation and one where each couple of values of conductivity and permeability do not cause condensation. The axes of the figure are the index of conductivity, the actual thermal conductivity of the insulation layer divided by the conductivity used in the design stage, and the index of water vapor permeability, the actual water vapor permeability of the insulation layer divided by the permeability used in the design stage.

Another example of performance limits is shown in figure 5. It shows how the indoor operating temperature is influenced by the change of insulation layer conductivity.

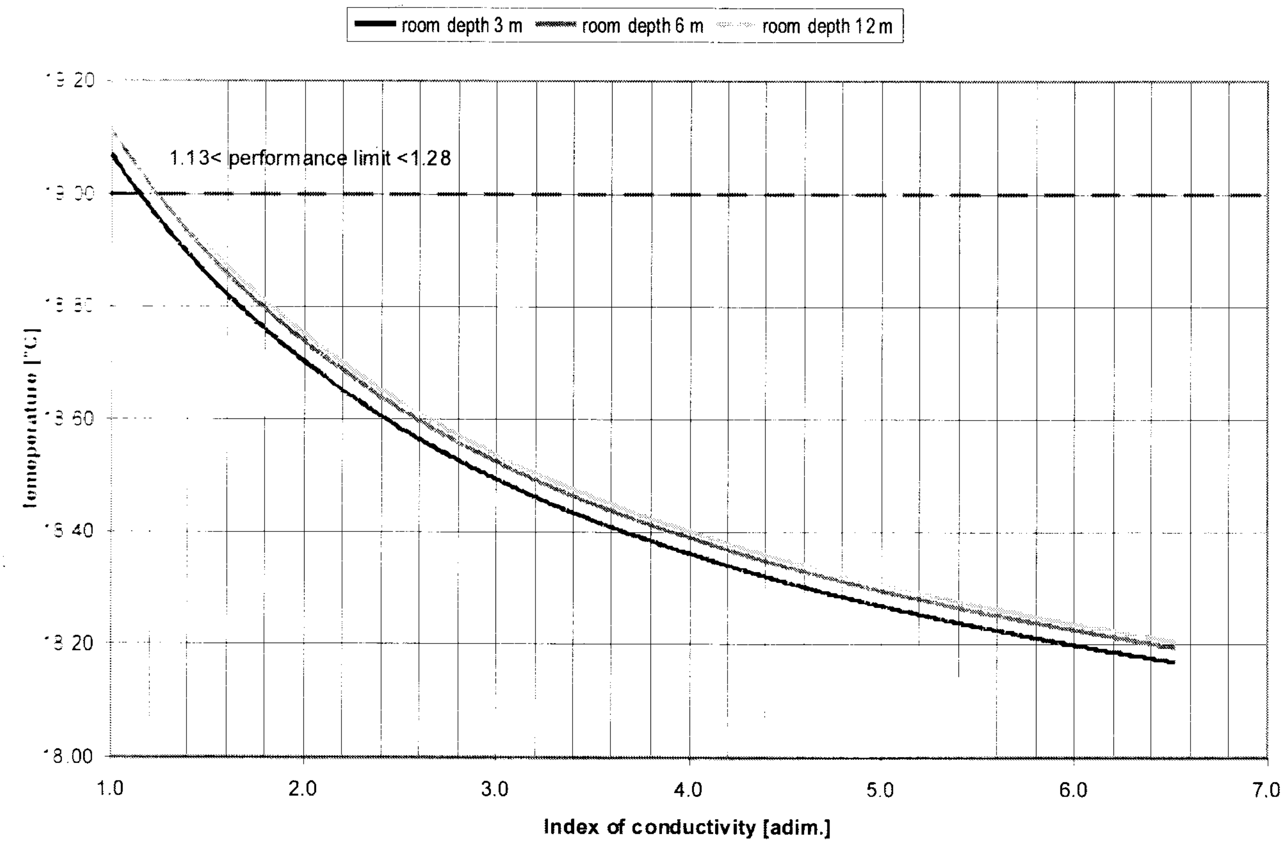


**Figure 5 – Operating temperature as a function of insulation layer conductivity**

The intersection between the curve of indoor operating temperature and the line of 19 degrees (minimum stated by Italian law) gives the performance limit for insulation layer conductivity. Indoor operating temperature depend also on hypotheses on room dimensions, in the figure three curves of operating temperature are computed for an Italian standard single bedroom (room depth 3 m), for a room with a double depth and for one with a 4 times depth. It can be seen that the influence of room dimensions on operating temperature is small.

The comparison between performance limits and the behavior of performance characteristics of materials during the life of a components, obtained from experimental tests (see Maggi *et al.* 1999) give us information on service life of the component in actual condition of use, i.e. its estimated service life.

In order to gain information on the influence of errors during different stages of the building process on building performance calculations a risk analysis was performed. A probability function with a triangle distribution (maximum error ±10%) was given to every value of the characteristics and a Montecarlo simulation was performed on both external walls under test.

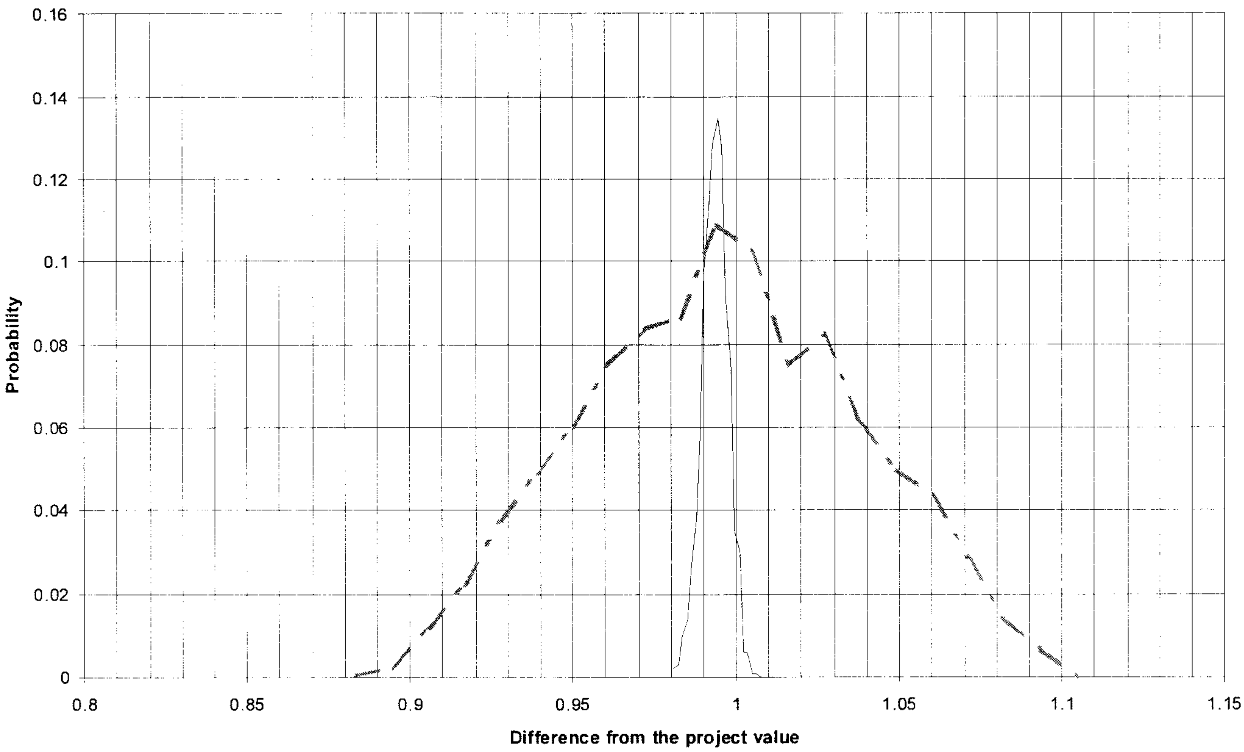


**Figure 6 – Control of thermal inertia in winter condition: results of Montecarlo simulation**

The result of the Montecarlo simulation is to measure the probability that the actual indoor climate will differ from the calculated one because of error in the design stage (choosing wrong values for material and climate characteristics) or the construction stage (choosing wrong values for the space characteristics). An example of results gained is shown in 'Fig. 6'. The figure, obtained with one thousand of Montecarlo simulation, shows the probability of a difference between the actual air temperature decreasing during winter night and the calculated one for both of the components of this test application (using the same space and climate conditions). It is important to notice how component 1 (the section of component 1 is show in Figure 2) is less influenced by errors then component 2 (dash - dot line).

## 3 CONCLUSIONS

Service life prediction has to be based on the solid pillar of performances (see service life definition in ISO 15686). Even if the Reference Service Life of a component is known it is hard to get to known its Estimated Service Life: methods to obtain ESL from RSL are still objects of scientists researches. The factor method seems to simplify too much the complexity of factors affecting degradation processes. Probabilistic methods are too expensive to be use in current practice, example of application on real engineer project have been done only on large-scale project. Engineering methods seem to be the right compromise solution, but literature lacks of example of this kind of method.

Performance limits methods directly links service life to performances and it is as easy to use as an engineering method. The application of the method presented on other project is strictly related to the spreading of knowledge on the behavior of performance characteristics of materials over time.

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# Development Of Damage Test Apparatus By Traffic For Car Deck Waterproofing Membrane Systems

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**Summary**: **A new type of a membrane system that combines a water proofing membrane and a car parking surface has been widely used in large shopping centres and car parking buildings in Japan. However, many failures caused by heavy traffic in the new membrane systems have appeared according as wider application of them occurs.An evaluation of performance of the new systems before actual application, in particular resistance to mechanical failures by heavy traffic, is needed to avoid unexpected failures in their service life.In the study, a test apparatus to reproduce damage by traffic was developed in the laboratory. Tests were carried out for several kind of car deck waterproofing membrane systems. Similar damage to those observed in the actual car deck membrane systems appeared in some specimens. It is concluded that the developed apparatus is able to reproduce the damage in a laboratory and is considered to be helpful for evaluating durability of them by traffic.**

**Keywords.** **car deck, waterproofing membrane system, test apparatus, damage modeling, traffic loading**

## 1 INTRODUCTION

A significant number of large shopping centers and facilities in which a car park is located on floors above the ground floor have been constructed in Japan. The surface of such floors therefore must be waterproof and have the functionality to endure repeated traffic loading. Traditional waterproofing membranes can not withstand heavy traffic loads by themselves and need a protective layer such as a concrete or asphalt-concrete pavement when applied to the deck of a car park. Such protective layers add additional loads to the structure and increase the cost of construction. There is therefore a significant demand for light membrane systems that do not need heavy protection layers.

Many car deck waterproofing membrane systems that combine a waterproofing membrane with a suitable surface have been developed and are widely implemented.However, damage caused by heavy loading is often observed. Hence, it is important to evaluate the performance of new systems before commercial application if in-service failures are to be avoided.

The purpose of this study is to develop a method for the evaluation of the performance of car-deck surfaces. An investigation of the damage to the decks of car parks due to traffic loadingwas performed. The results of the investigation were used in the development of test apparatus for reproducing such similar damage.

## 2 INVESTIGATION OF DAMAGE TO CAR DECK WATERPROOFING MEMBRANE SYSTEMS

### 2.1 Outline of investigation

Investigation of the damage to 46 car park decks in the Tokyo area was performed. The items of the investigation are as shown in Table 1.

**Table 1 Items for Investigation**

| | |
|---|---|
| Building | Name, Location, Date of completion, Size of the building, Type of structure |
| Parking area | Capacity, Daily number of users, Design of parking area |
| Damage | Location, Description and size of damage, Estimated cause |

## 2.2 Observed damage

Most damage only appeared on parking decks. However, it was classified as that only observed on parking decks or only observed on ordinary waterproofing membrane systems. The former was further subdivided into three groups as shown in Table 2.

The two types of damage most commonly observed were abrasion and separation as shown in Pictures 1 and 2. Abrasive damage is caused by wheel loading and separation damage occurs when the membrane is separated from the concrete substrate and is ruptured.

**Table 2 Types of damages**

| | | |
|---|---|---|
| Damage only observed to car deck | Abrasion | Abrasion of surface, Removal of gravel, Exposure of concrete |
| | Separation | Separation of membrane from substrate, Rupture of membrane |
| | Others | Cracking of membrane, Scratching due to contact with underside of car, Scratching by chains |
| Common damage observed on waterproofing membrane | Blistering, Delamination, Cracking, Splitting by cracking in substrate | |

## 2.3 Results

The relation between the total number of cars using the deck and the service interval of the deck system is denoted in Fig.1 by black marks for damaged systems and white marks for undamaged ones. The total number of cars was estimated by multiplying the average daily number of cars by the service period.Most occurrences of damage were observed on decks using new waterproofing membrane systems without a protective layer and subjected to loading from several hundred thousand cars over the service life. The service lives of such decks were shorter than traditional decks fitted with protective layers.

Fig.2 shows the locations where damage occurred to the car decks and the total number of occurrences of each type of damages. With regard to damage caused by abrasion, most occurrences were observed in curved traffic lanes, followed by slopes and straight lanes. The results also suggest that new car deck waterproofing membrane systems without protective layers are not durable compared with older surfacing techniques. With regard to damage caused by separation, although the number of occurrences is significantly less than that of abrasion, such damage frequently occurred on straights, curves and slopes.

## 3 ESTIMATION OF FORCES

**Picture 1 Abrasion**

**Picture 2 Separation from the substrate**

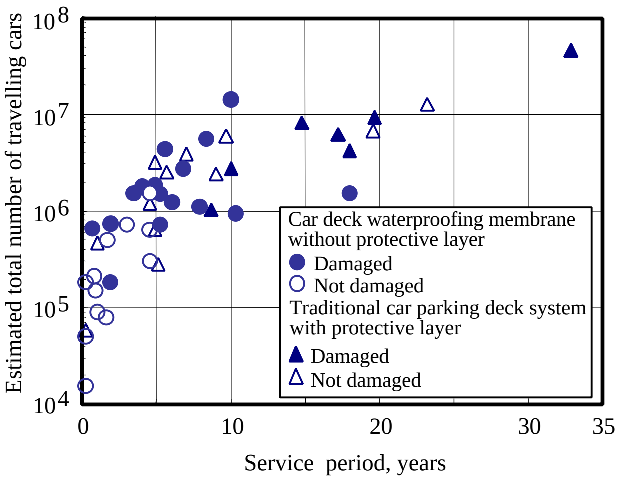



**Fig.1 Relation between total number of users and service period of deck system**

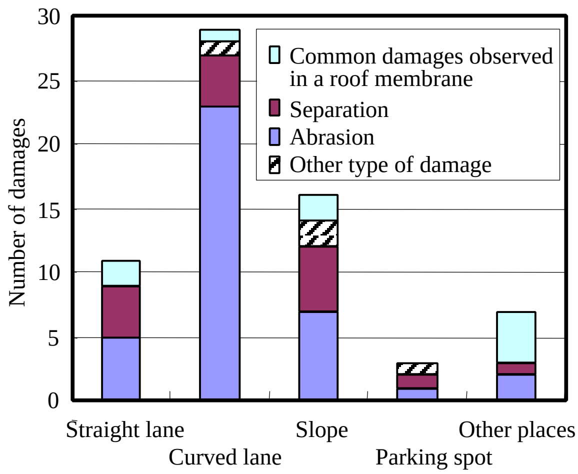



**Fig.2 Location and frequency of damage**

### 3.1 Force transmitted through the wheel

Traffic loading is the load transmitted by the car tire, the most extreme case of which is considered to occur when the direction of motion is changed. When a car changes direction, lateral forces occur in the wheel as shown in Fig.3. As the rolling direction of the wheel does not accord with the direction of travel of the car, slippage, which is denoted the slip angle, is induced between the wheel and the deck. Thereby, a force opposing the centrifugal force of the car, which is called the cornering force, appears in the direction orthogonal to the motion of the car. When the slip angle is small, it approximates the lateral force.

The lateral force is distributed in the zone of contact between the tire and the surface. It also produces a moment called the self-aligning torque around the centre of the contact area, because the centre of the resultant lateral force is to the rear of the centre of the contact area. The surface is, therefore, subjected to two reaction forces.

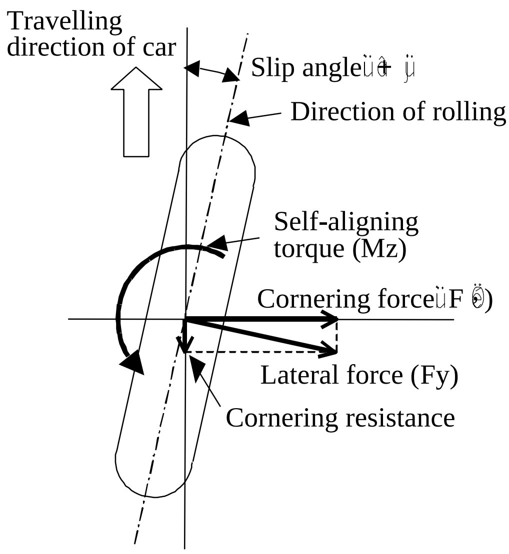



**Fig.3 Forces encountered during cornering**

### 3.2 Quantitative estimation of loading on a parking deck

The lateral forces and self-aligning torque can be calculated using the extended Fiala's equation.The lateral force Fy is considered as the sum of lateral forces in a non-slip area and the slip area on a tire. When the boundary of the two areas is given as $l$h, Fy is expressed by equation (1). The cornering force F'y and the self-aligning torque Mz can be deduced from the equation and are expressed by equations (2) and (3) respectively.

$$F_y = C_y l_h^2 \beta \left[ \frac{1}{2}\tan\beta - \left( \frac{\mu}{C_y} + \frac{4l^2}{3\mu^2 G'_y} \right) \frac{F_y}{l} \left( \frac{1}{2} - \frac{l_h}{3l} \right) \right]$$

$$+ \frac{n+1}{n} \frac{2^n F_z \mu_d}{l^{n+1}} \left[ \left( \frac{1}{2} \right)^n \left( l - l_h \right) - \frac{1}{n+1} \left\{ \left( \frac{l}{2} \right)^{n+1} - \left( l_h - \frac{l}{2} \right)^{n+1} \right\} \right] \quad (1)$$

$$\text{where } \mu_d = \mu_0 - a \frac{l V}{l_h - l} \sin\theta \quad (a\text{: constant})$$

$$F'_y = F_y \cos\theta \quad (2)$$

$$M_Z = C_y l_h^2 \beta \left( \frac{l_h}{3} - \frac{l}{4} \right) \tan\beta + \left( \mu + \frac{4l^2 C_y}{3\mu^2 G'_y} \right) \frac{F_{\theta}\beta\, l_h}{4l^2} \left( l_h - l \right)^2$$

$$+ \frac{n+1}{n} \frac{2^n F_z \mu_d}{l^{n+1}} \left[ \frac{1}{2} \left( \frac{l}{2} \right)^n \left\{ \left( \frac{l}{2} \right)^2 - \left( l_h - \frac{l}{2} \right)^2 \right\} \right.$$

$$\left. - \frac{1}{n+2} \left\{ \left( \frac{l}{2} \right)^{n+2} - \left( l_h - \frac{l}{2} \right)^{n+2} \right\} \right] \quad (3)$$

Substituting each value in Table 3 into these equations, the cornering force and self-aligning torque for slip angles from 0 to 12 degrees were calculated for a concrete pavement and a urethane rubber membrane system with a non-slip gravel layer as shown in Fig.4. The cornering force increases with slip angle, is affected by load and is higher at a load of 4.90 kN than at 2.94 kN.With regard to the self-aligning torque, the value also increases with slip angle. However, an upper limit is observed and the value decreases at larger slip angles. The cornering force and self-aligning torque are higher on the urethane rubber membrane system than on the concrete pavement. This is because the gravel adhered to the surface of the urethane rubber membrane increased the dynamic friction between the membrane and the tire.

**Table 3 Adopted values for calculating cornering forces and self-aligning torque**

| | |
|---|---|
| G'y: Lateral Stiffness of the tire | 24.5N/mm$^2$ |
| Cy: Lateral spring constant of the tire | 8.4N/mm$^2$ |
| Vs: Speed | 8km/h |
| ?: Effective rolling radius of the wheel | 270mm |
| l: Length of tire in contact with the deck | 100mm |
| ? : Width of tire in contact with the deck | 120mm |
| Fz: Applied load | 4.90kN |
| a: Slip angle | 0-12deg |
| n: Constant obtained from pressure distribution in zone of contact | 4 |
| a: Coefficient | 0.05h/km |
| d: Material constant of surface | 24 |
| $\mu_0$: Coefficient of dynamic friction | 0.8(for concrete)<br>1.1(for Urethane rubber with non-slip layer) |

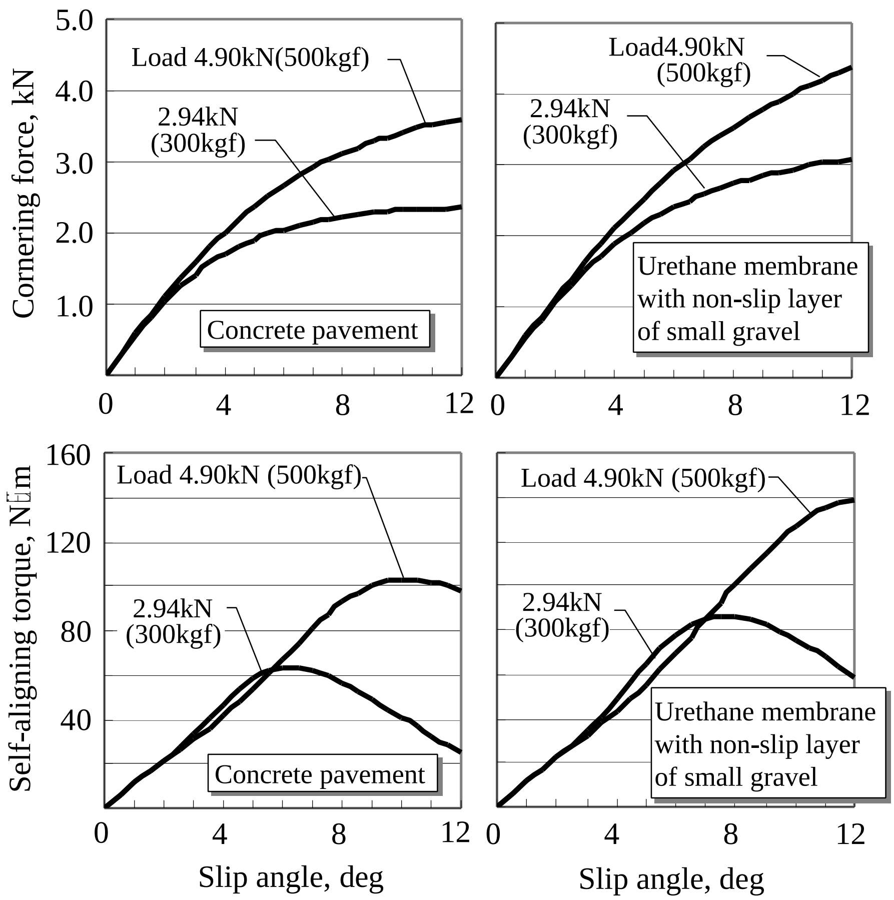



**Fig.4 Relation between forces in the tire and slip angle**

## 4 DEVELOPMENT OF TEST APPARATUS

### 4.1 Requirements

On the basis of the initial investigation, we developed an apparatus to reproduce damage similar to that observed on actual car deck waterproofing membrane systems. The design brief was based on passenger cars and a commercial car park facility. In particular, the apparatus should be able to model the behavior observed on curved lanes.

The requirements are therefore such that the apparatus should be able to approximate the forces encountered during typical loading of an actual car park and the damage should be recreated within a reasonable period of time.

### 4.2 Simplification of behavior of a wheel while a car changes direction

4.2.1 Motion of a wheel

It is important to simplify the complicated behavior of a wheel during turning operations. All wheels skip according to the slip angle when a car changes direction. We therefore adopted a simple mechanism in which the wheel moves in a constant direction. In this way, the behavior can be replaced by a simple straight motion as shown in Fig.5. A slip angle of 4 degree was adopted, which is the maximum value during ordinary turning operations in a car park.

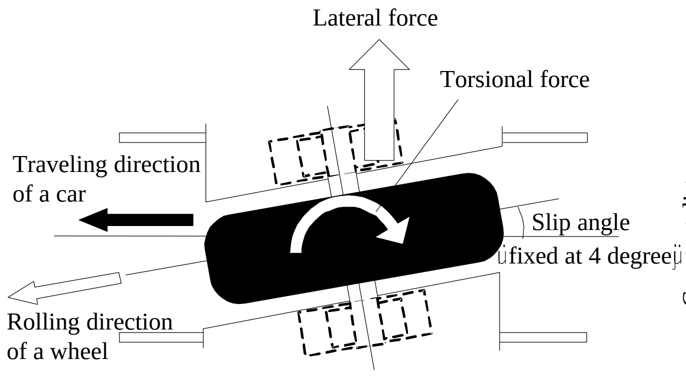



**Fig.5 Simplified mechanism to reproduce conditions during cornering**

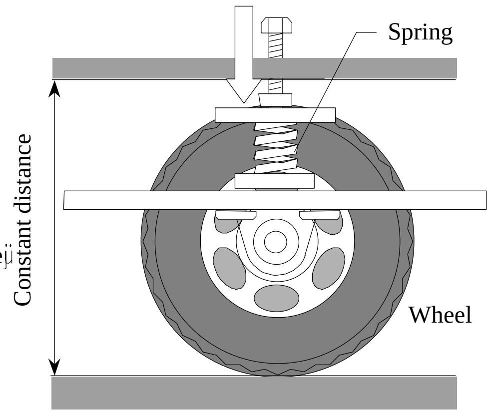



**Fig.6 Mechanism for maintaining wheel loading**

4.2.2 Traveling speed

A maximum speed of 8 km/h, which is an estimate of the average speed in the parking areas in buildings,was used.To obtain this speed in the small apparatus, the test wheel and surface were simultaneously moved in opposite directions.

4.2.3 Loading

The weight of a medium size car is estimated to be 2 tones, so each wheel transmits a load of 4.9 kN. The mechanism for applying the load via a spring-loaded wheel is shown in Fig.6.

### 4.3 Test apparatus

Considering the above requirements, a test apparatus was constructed in accordance with the design shown in Fig.7. A radial tire of diameter 635 mm and inflated to 0.2 MPa was attached to the wheel rim.

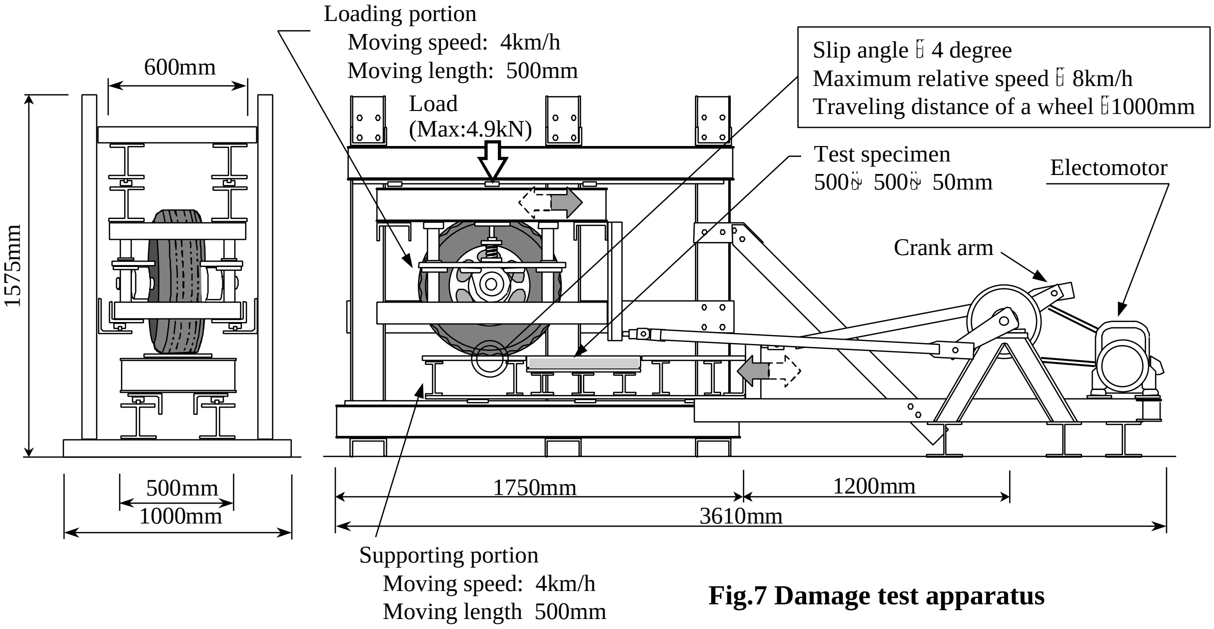



**Fig.7 Damage test apparatus**

### 4.4 Performance of the apparatus

The vertical and lateral forces were monitored to ensure that they remained within the intended levels. Monitoring of the self-aligning torque was not performed because it is negligible as shown Fig.4 and was considered not to be a cause of damage.

#### 4.4.1 Force Measurement

A device as shown in Fig.8 was made to monitor the forces. In order to measure the vertical force, a load cell was fixed under the steel plate supporting the surface on which the wheel travels. A second load cell was fixed horizontally under the next steel plate to measure the horizontal force. Both cells were capable of measuring loads of 9.8 kN.A concrete pavement was used as the standard surface, and two concrete panels (300 mm × 300 mm × 50mm) were fixed to the apparatus with bolts.

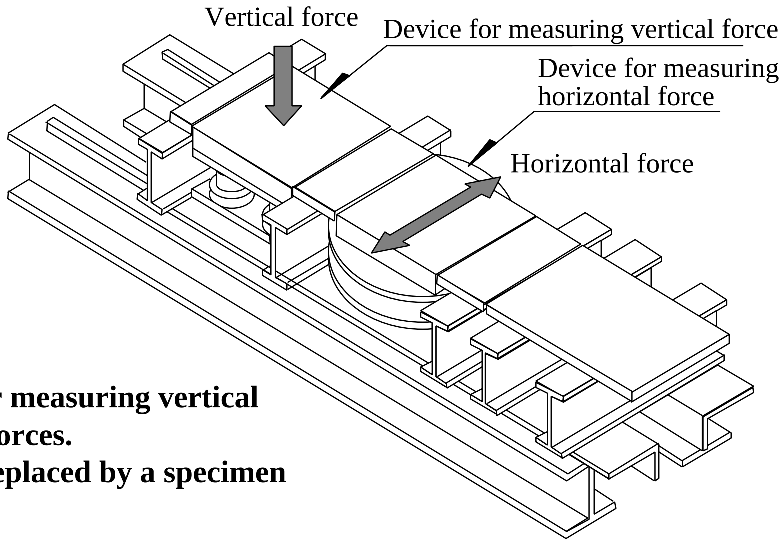



**Fig.8 Position of devices for measuring vertical @@@force and horizontal forces. (The latter device is replaced by a specimen during the test.)**

#### 4.4.2 Measurement procedure

The devices were installed at the two positions in the direction of wheel travel. The measurement procedure was as follows. First, the wheel was stopped on the center of the surface specimen to measure the vertical force and the load adjusted by changing the length of the compression spring between the wheel axis and the upper beam in the loading portion. The wheel was moved at 8 km/h relative to the test surface, and the lateral force was measured.

#### 4.4.3 Estimation of performance of the apparatus

The relation between applied load and cornering force for the concrete pavement is shown in Fig.9. The cornering force increased with applied load. The calculated value is also shown in the same figure. The measured cornering force agrees approximately with the calculated one, and therefore the apparatus is considered to reproduce actual loading conditions.

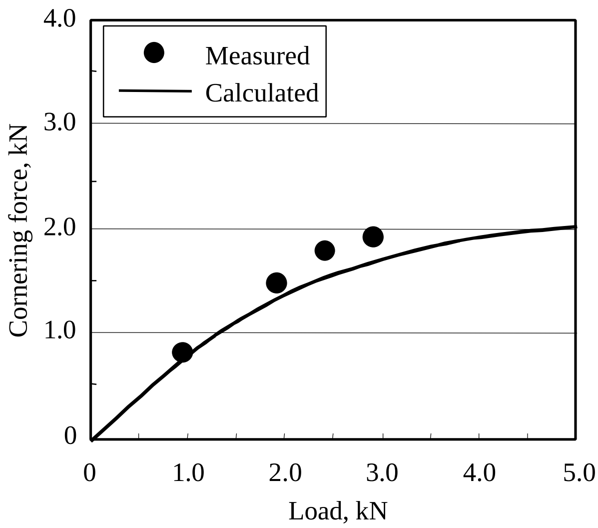



**Fig.9 Comparison between measured and calculated cornering forces**

## 5 TESTS ON COMMERCIALLY AVAILABLE WATERPROOFING MEMBRANE SYSTEMS

### 5.1 Test sample and specimen

The specifications of the test samples are shown in Table 4. Sample No.1 is a low-grade specification upon which damage is expected to appear during early stages of the test. With regard to samples No.3 and 4, the non-adhered areas were provided as shown in Fig. 10 to simulate the occurrence of separation damage because damage is frequently observed in poorly adhered areas in actual car decks.

With regard to the size of each specimen, a travelling path of at least 500mm is adopted. The samples were applied on steel reinforced concrete panel of 50mm thick, and were cured at room temperature for one week and furthermore stored in an oven at 50 ? for 3 days to ensure sufficient curing.

**Table 4 Test car deck waterproofing membranes**

| Sample | Materials | Adhesion to concrete substrate | Applied quantity | Non slip layer |
|---|---|---|---|---|
| No.1 | Urethane rubber (Soft type) | Fully bonded | 1.2kg/m$^2$ | - |
| No.2 | Urethane rubber (Hard type) | Fully bonded | 1.2kg/m$^2$ | - |
| No.3 | Urethane rubber (Hard type) | Prepared non-adhered area | Upper layer 1.2kg/m$^2$ | Small gravel (2kg/m$^2$) |
| | | | Bottom layer 0.3kg/m$^2$ | |
| No.4 | Urethane rubber (Hard type) | Prepared non-adhered area | Upper layer 1.5kg/m$^2$ | Small gravel(5kg/m$^2$) |
| | | | Bottom layer 0.8kg/m$^2$ | |

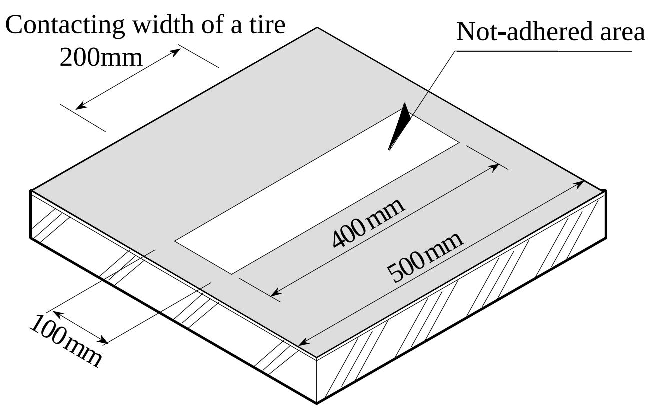



**Fig.10 Area not adhered to concrete substrate in sample 3 and 4**

### 5.2 Test procedure

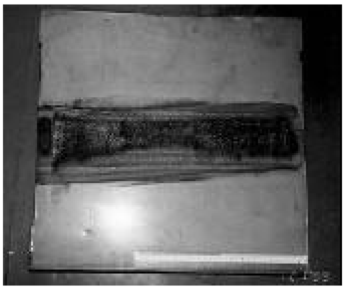
A load of 2.24 kN (300 kgf), which is lighter than actual loading, was applied as the preliminary stages of the study. The test was continued until damage was observed on the specimen, and stopped after 100,000 cycles when no damage was observed. Tire wear was monitored during the test, and the tire was immediately replaced when the low-tread indicator was observe d. A number of cycles and the details of the damage were recorded.

### 5.3 Test results and discussion

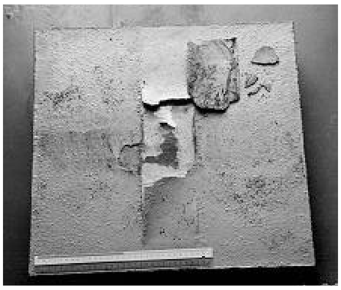
Table 5 shows the test results. Signs of abrasion as shown in Picture 3 were observed on sample No.1, which was manufactured from low-grade material, after 4000 cycles. Separation damage as shown in Picture 4 appeared on sample No 3. Even though the specification matched that of a commercial car deck waterproofing membrane system, splitting as shown in Picture 5 appeared in the membrane of No.4 along the edge of the non-adhered area. Abrasion and removal of the gravel was observed on all specimens with non-slip layers. All damages produced by the apparatus closely resembled that observed on actual car decks.

**Table 5 Test results**

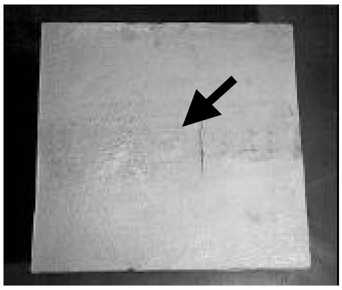
| Sample | Number of loading, cycles | Details of damage |
|---|---|---|
| No.1 | 4,000 | Abrasion of membrane |
| No.2 | 100,000 | Trace of tire traveling |
| No.3 | 1,000 | Separation from concrete substrate |
| No.4 | 100,000 | Splitting of membrane along the edge of non-adhered area |



**Picture 3 Abrasion of membrane**



**Picture 4 Separation of membrane from substrate**



**Picture 5 Splitting of membrane along the edge of non-adhered area**

## 6 CONCLUSIONS

The conclusions of the study are as follows.

1. The investigation of existing car parks revealed that new decking systems combined with a waterproofing membrane are susceptible to damage. The prominent modes of damage were abrasion of the membrane and separation of the membrane from the substrate.
2. The movement of a wheel during cornering can be modelled by moving a wheel in a straight motion at an angle slightly different to its direction of rolling. An apparatus to model the damage to car deck systems caused by repeated traffic loading was developed on the basis of this observation .
3. Although some of the specimens were intentionally weakened, the damage produced by the apparatus resembled that observed on actual car deck membrane systems.

## 7 ACKNOWLEGEMENTS

The authors would like to thank all the members of the research committee on car deck waterproofing membrane systems for their help, Dr. Takamasa Mikami for his useful discussion and Mr.Hajime Ishii for designing the apparatus.

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# Predicting The Service-Life Of Natural Roofing Slates In A Scottish Environment

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**Summary**: **Natural roofing slate, the traditional roofing material of most 18th and 19th century towns and cities of Scotland has performed effectively for over 150 years. Yet in spite of their reputation for durability, many Scottish slates failed to comply with the requirements set by the British standard for roofing slate (BS680). This inability of the British and other standards to predict the performance of a slate led to this research into alternative methods of assessing quality. All national standards including the new European standard use initial water absorbency as a significant factor in determining quality of a slate. However this research shows that there is poor correlation between this factor and life expectancy and that of greater significance is the rate of increase in water absorbency during weathering. This factor was determined for a range of roofing slates by measuring water absorbency before and after experimental weathering and used as a measure of its durability. Results were calculated relative to that of a Welsh standard, which was assigned a life expectancy of 100 years. It was found that results agreed with their generally accepted reputation; all the Welsh samples have life expectancy values similar to that of the standard, the Cumbrian slates have values of over 100 years and the Spanish slates have values of approximately 50 years. To estimate longevity in absolute terms, the relationship between experimental and natural weathering was determined by comparing the corresponding rates of increase in water absorbency for slates from the same source. However because the environment is a major factor in weathering, this tentative calibration can only be applied to slates weathered in a similar climate to that of Scotland**

**Keywords. slate, durability, whole life costing**

## 1 INTRODUCTION

The durability of a roofing slate depends on many factors both inherent and external. It depends on the properties of the material such as its solubility, porosity and permeability and also on the nature of the environment particularly the climatic and anthropogenic influences. Concentrating on differences in their inherent properties the aim of this research is to devise a method of predicting ***relative*** durability of different slates based on carefully controlled experiments.

## 2 BACKGROUND

In an ASTM (American Society for Testing and Materials) study of roofing slates, the chemical and physical properties of used slates which had been exposed from 12 to 131 years were compared with those of freshly quarried slates from the same sources (Kessler and Sligh 1932). In this way it was found that the most significant changes due to natural weathering were loss of strength and increase in water absorbency. The study also showed that the same chemical and physical changes can be replicated experimentally by subjecting the slate to repeated cycles of wetting-and-drying. Based on this work the American standard for slates incorporated a water absorption test and a wetting-and-drying test. Given the plethora of methods used to test slates, it is significant that the new European Standard prEN12326, the British Standard BS680 and all other national standards also include water absorption and wetting-and-drying procedures.

### 2.1 Water absorption test

Water plays an important role in both the physical and chemical weathering processes of all natural building materials. Most national standards and the European standard prEN12326 adopt a simple immersion test whereby the amount of water absorbed by a dried slate in 48 hours at ambient temperature is measured. The Building Research Establishment (BRE) found that it took nearly a month for the amount of water absorbed by this method to reach equilibrium (Watkins 1934). However it was found possible to accelerate this process and attain equilibrium by refluxing (boiling) the sample for 48 hours. This was the method adopted by the British Standard for slate BS680. As a result of boiling the sample, instead of simply immersing it in water at ambient temperature, the figures for water absorption tested according to BS680 are usually substantially higher than those obtained using the method prescribed in the ASTM, prEN12326 and other standards.

### 2.2 Wetting-and-drying test

The wetting-and-drying test subjects a slate to repeated cycles of immersion in water at ambient temperature and drying in an oven at approximately 105°C. The procedures prescribed by both the British BS680 and EU prEN12326 standards for this test are essentially the same. Because the conditions closely reflect those experienced by a slate on a roof in a temperate climate, it is the method chosen to experimentally weather slates in this research. In addition changes in the properties such as increase in water absorption and loss of strength mirror those observed in naturally weathered slates.

### 2.3 Durability of slate

Prior to the establishment of the British Standard (BS680), BRE carried out a wide range of tests and correlated the results obtained with the reputation of the slate in question. The tests deemed to give the best correlation were then incorporated into the Standard. These were (i) the water absorption test, (ii) the acid test and (iii) the wetting-and-drying test. Slates tested were either passed or failed and no attempt was made to differentiate between different quality of slates. In the case of water absorbency, the limit was set pragmatically at 0.3% in an attempt to exclude slate of perceived poor quality. However it was recognised that this also excluded slates with a proven track record

> *…some Scottish slates of good reputation fail to meet the requirements of the (British) Standard for no other reason than that their water absorption exceed by a small margin the present limiting value of (0.3 per cent.).* Note C 330 BRE archives unpublished).

In contrast, the European standard does assign one of several grades to slates based on performance in a wide range of tests, however no attempt is made to relate these grades to their estimated longevity. The ASTM standard is the only one which estimates the life expectancy of a slate. Based on initial water absorption and depth of softening when exposed to acid vapour, this standard classifies slate into one of three grades, to each of which is assigned a corresponding life-expectancy value (Table 1).

**Table 1 Life expectancy based on water absorption and depth of softening on exposure to acid vapour. ASTM data**

| | *Modulus of Rupture* | | *Water Absorption* | *Depth of softening* | | *Life expectancy* |
|---|---|---|---|---|---|---|
| | *psi* | *MPa* | *%* | *inch* | *mm* | *years* |
| Grade $S_1$ | 9000 | 62 | <0.25 | <0.002 | <0.05 | 75-100 |
| Grade $S_2$ | 9000 | 62 | <0.36 | <0.008 | <0.20 | 40-75 |
| Grade $S_3$ | 9000 | 62 | <0.45 | <0.014 | <0.36 | 20-40 |

However recent research (Walsh 1999) found no relationship between **initial** water absorbency values as used in the ASTM standard and the reputation of a slate. For example a Welsh slate from Pen yr Orsedd Quarry has a water absorption of 0.24% which is higher than that of a Spanish slate from Solana de Forcadas Quarry at 0.15%. Yet a Welsh slate is expected to last over a hundred years compared to a Spanish slate which is estimated to last approximately thirty years (Maol in Builder 1995). Instead of concentrating on initial values, this research assesses the effect of weathering by measuring properties such as water absorbency at intervals during experimental weathering, using the wetting-and-drying procedure described above, in order to determining their **rate of increase**. The higher the rate of increase the more vulnerable the slate is to weathering. The profile of how a given property changes with the degree of experimental weathering is referred to as its **weathering pathway**.

## 3 METHODOLOGY

To assess the durability of a slate it is necessary to measure its properties before and after a period of experimental weathering. Because of the natural variation even within a single slate, measurements of changes in those properties which are assessed using destructive tests, requiring a different test piece for each measurement, are less precise than those assessed using non-destructive methods. In addition properties measured using destructive tests give only a single figure on the amount of change due to weathering and no information on how the change took place during the procedure i.e. its weathering pathway. Therefore the principal property used in assessing the effects of weathering is water absorption which is measured using a non-destructive test, enabling the same sample to be tested repeatedly at intervals during experimentally weathering. This gives a relationship between water absorption and the degree of experimental weathering on which to base a mathematical model, which can then be used to assess the durability of a slate by extrapolation to a predetermined limit. If possible this ***limit of water absorption*** should be set as that determined when the slate shows signs of failure.

Ideally the increase in water absorption due to experimental weathering would provide a characteristic profile, which could then be compared with that observed in similar slates that have been naturally weathered and the experiment thereby "calibrated". Then, given a suitable limit of water absorption, the life expectancy of the slate could be determined. To test this hypothesis, three samples were experimentally weathered by subjecting them to repeated cycles of wetting-and-drying to determine whether it was possible to detect significant changes in their properties within a reasonable timeframe and thereby establish a procedure for estimating the relative durability of different slates. The slates chosen were:

1. I/K-3 Killaloe slate from Killoran Quarry, Portroe near Nenagh, Ireland
2. W/F-8 Ffestiniog slate from Oakley Quarry, Ffestiniog, North Wales
3. S-1 Spanish slate from Solana de Forcadas Quarry, Spain

Data for naturally weathered slates from the Killoran and Oakley quarries were available enabling comparisons to be made between the effects of natural and experimental weathering. Having developed the optimum procedure for assessing durability, a range of different slates were tested, their relative durabilities estimated and where possible the results compared with their reputations.

Four test pieces, 50mm square, were cut from each slate, experimentally weathered and changes in the following properties assessed:

- Visual signs of weathering such a flaking, discoloration etc. were monitored throughout the experiment.
- Weight and thickness measurements were made at frequent intervals. To minimise the variation due to a rough surface, thickness measurements were always made at the same four locations on each of the test pieces.
- Water absorbency; two of the test pieces of each slate were tested by refluxing as prescribed by BS680 and the remaining two pieces by simple immersion as prescribed by prEN12326. The tests were repeated four times at increasing intervals until approximately 100 - 150 cycles had been carried out.

### 3.1 Visual signs of weathering

I/K-3 showed no visual sign of weathering throughout the experiment. Both W/F-8 and S-1 developed brown spots on the surface after a hundred cycles of wetting-and-drying but no other visual signs of weathering were observed.

### 3.2 Weight and thickness

No significant pattern of weight loss in the slates used in this research emerged, however supplementary work (Walsh in press) suggests that good quality slates show a small and continuous weight loss, while some poor quality samples increase in weight with weathering.

A systematic trend in thickness measurements was observed, in that all the slates tested increased in thickness during weathering. Although the actual increments were small, when analysed statistically using a Student's t-test the changes were found to be significant. These results are shown in Table 2 and those for W/F-8 are plotted against length of weathering to show how the rate of increase in thickness increased after a hundred cycles of wetting-and-drying (Fig.1).

**Table 2 Increase in thickness due to experimental weathering**

| *Thickness (mm)* | *Wetting-drying cycles* | *Mean* | *Standard deviation* | *95% Confidence Interval* | *Percentage Increase after 100 cycles* | *Student's t-test* |
|---|---|---|---|---|---|---|
| I/K-3 | 0 | 5.900 | 0.163 | ±0.08 | | |
| | 127 | 6.025 | 0.191 | ±0.09 | 1.7 | 0.06 |
| W/F-8 | 0 | 3.688 | 0.096 | ±0.04 | | |
| | 123 | 3.763 | 0.086 | ±0.04 | 1.7 | 0.02 |
| | 140 | 3.875 | 0.061 | ±0.04 | | |
| S-1 | 0 | 6.663 | 0.072 | ±0.04 | 3.1 | |
| | 97 | 6.775 | 0.068 | ±0.04 | | 0.00 |

### 3.3 Water Absorbency

The results of water absorption tests are given in Table 3 and summarised in Figs. 2 and 3. It can be seen from these results that the slates tested showed a measurable increase in water absorption due to experimental weathering and that the results are reproducible. In general the rate of increase in absorbency increased with the degree of weathering. This is especially clear in the case of S-1, whose rate of increase in water absorbency increased significantly after an initial period of little change.

Regressional analyses of the data in Table 3 were carried out using both linear and exponential models and the closeness of fit assessed by determining the corresponding coefficients of determination. Poor fits ($R^2 \leq 0.4$) were obtained for those samples tested according to prEN12326 (Fig.2), which is interpreted as being due to the small increases in weights as compared to the imprecision inherent in weighing wet samples. To get a better fit by this method would require extending the already lengthy weathering procedure. With one exception (I/K-3) those samples measured using the BS680 method of refluxing have coefficients of determination ($R^2$) of approximately 0.8 indicating a good fit (Fig. 3). In the case of I/K-3 a poorer fit was obtained ($R^2$ =0.63) due to the very low rate of increase in absorbency. Although the coefficients of determination were approximately the same for both linear and exponential models, the latter were slightly higher in every case. This suggests that the exponential model is a truer representation of the effect of weathering, which is substantiated by other observations in both experimental and natural weathered slates.

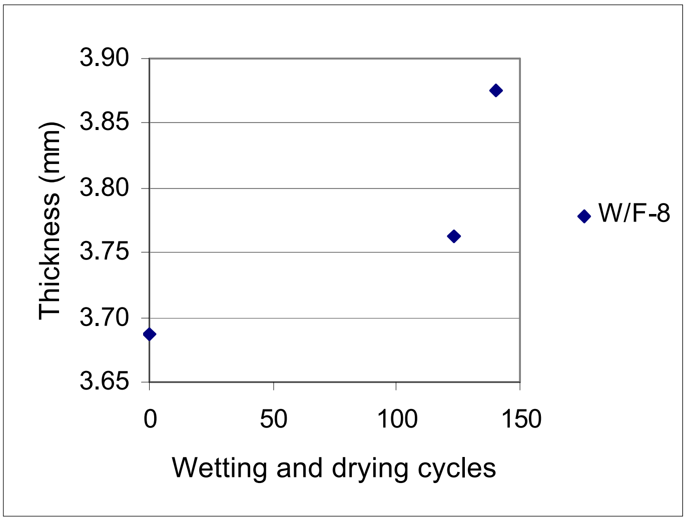



**Fig.1 Increase in thickness of a slate due to experimental weathering**

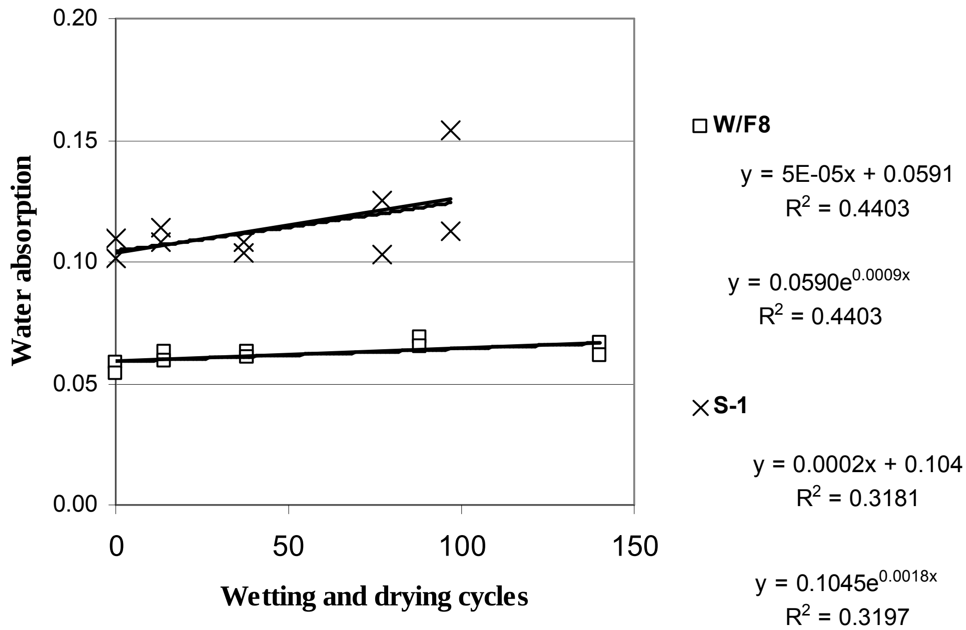
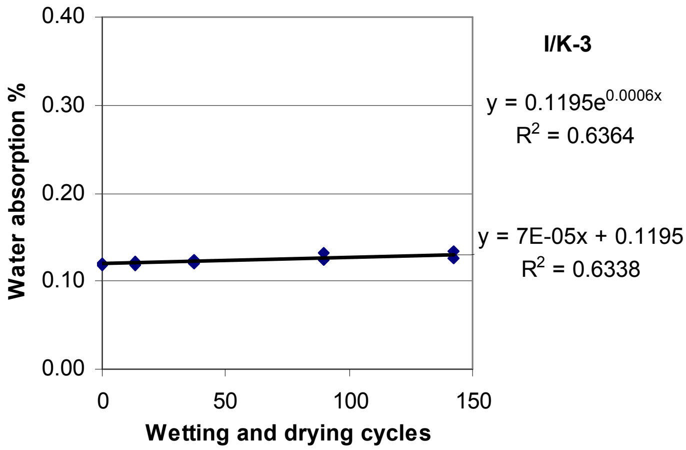
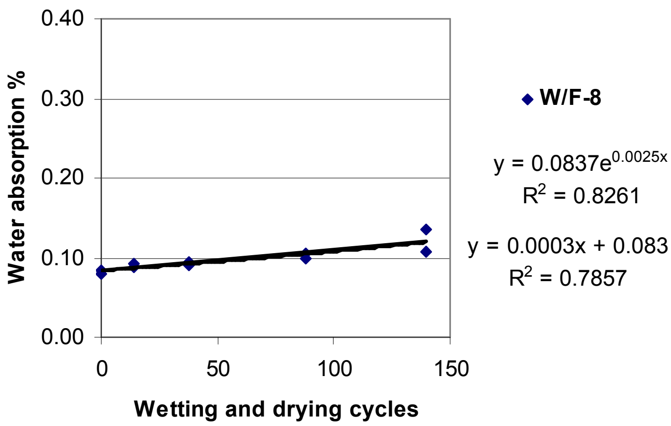



**Fig. 2 Experimental weathering of Welsh and Spanish slates; water absorbency tested according to prEN12326**

**Table 3 Increase in water absorbency due to experimental weathering**

| | | | | | |
|---|---|---|---|---|---|
| I/K-3 | | | | | |
| No of cycles | 0 | 13 | 24 | 53 | 52 |
| Cumulative Total | 0 | 13 | 37 | 90 | 142 |
| Water absorption (%) | 0.1174 | 0.1177 | 0.1204 | 0.1236 | 0.1255 |
| BS680 | 0.1212 | 0.1224 | 0.1242 | 0.1314 | 0.1331 |
| Mean BS680 | 0.1193 | 0.1200 | 0.1223 | 0.1275 | 0.1293 |
| prEN12326 | 0.0929 | 0.0959 | 0.0951 | 0.0959 | 0.0962 |
| | 0.0889 | 0.0938 | 0.0928 | 0.0917 | 0.0901 |
| Mean prEN12326 | 0.0909 | 0.0948 | 0.0939 | 0.0938 | 0.0931 |
| ***W/F8*** | | | | | |
| No of cycles | 0 | 14 | 24 | 50 | 52 |
| Cumulative Total | 0 | 14 | 38 | 88 | 140 |
| Water absorption (%) | 0.0804 | 0.0875 | 0.0897 | 0.0997 | 0.1077 |
| BS680 | 0.0839 | 0.0921 | 0.0943 | 0.1046 | 0.1357 |
| Mean BS680 | 0.0821 | 0.0898 | 0.0920 | 0.1022 | 0.1217 |
| prEN12326 | 0.0583 | 0.0628 | 0.0628 | 0.0692 | 0.0665 |
| | 0.0541 | 0.0595 | 0.0608 | 0.0653 | 0.0616 |
| Mean prEN12326 | 0.0562 | 0.0612 | 0.0618 | 0.0673 | 0.0640 |
| S-1 | | | | | |
| No of cycles | 0 | 13 | 24 | 40 | 20 |
| Cumulative Total | 0 | 13 | 37 | 77 | 97 |
| % water absorption | 0.148 | 0.160 | 0.164 | 0.180 | 0.221 |
| BS680 | 0.152 | 0.162 | 0.166 | 0.178 | 0.219 |
| Mean BS680 | 0.1500 | 0.1611 | 0.1654 | 0.1788 | 0.2201 |
| prEN12326 | 0.1013 | 0.1084 | 0.1035 | 0.1033 | 0.1128 |
| | 0.1098 | 0.1143 | 0.1084 | 0.1250 | 0.1537 |
| Mean prEN12326 | 0.1056 | 0.1113 | 0.1060 | 0.1141 | 0.1332 |



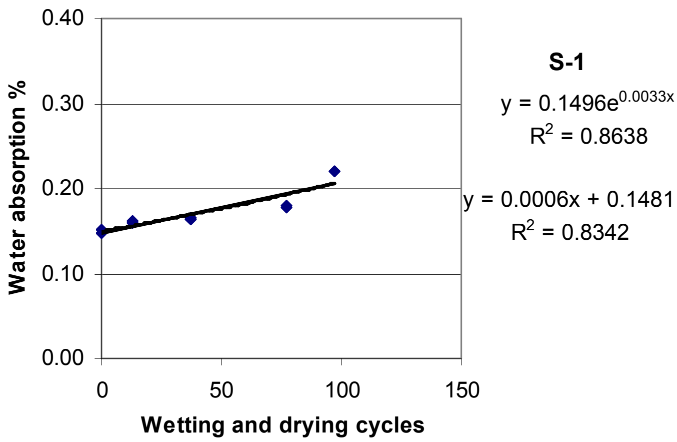




**Fig. 3 Experimental weathering of slates tested according to the BS680 procedure (a) Irish, Killaloe I/K-3 (b) Welsh, Ffestiniog W/F-8 and (c) Spanish, Solana de Forcadas S-1 samples.**

## 4 DISCUSSION

To estimate the durability of a slate, it is necessary to choose a suitable mathematical model which best represents the experimental data. The model chosen is then extrapolated in order to make predictions on performance in the future. Prior knowledge suggests that the deterioration of a slate due to weathering increases with time and that the most appropriate mathematical model is exponential. However in the last section, it was found that linear mathematical models fit the data almost as well as their exponential equivalents due to the fact that the early part of the exponential curve approximates to a straight line. Therefore it should be possible to differentiate between different quality of slates based on the differences in the gradients of their linear models. The linear regressional line for S-1 is

$$\mathbf{y = 0.0006x + 0.1481}$$

where **x** is the number of cycles of wetting-and-drying and **y** is the percentage water absorption (Fig.3c). This represents an initial water absorbency of 0.1481% and an initial rate of increase of 0.0006% per cycle. Comparing this with the linear regressional lines for I/K-3 and W/F-8 (Fig 3a and 3b), it can be seen that the rate of increase of water absorbency of the S-1 sample is approximately nine times that of I/K-3 (0.00007% per cycle) and twice that of W/F-8 (0.0003% per cycle). This demonstrates that it is possible to differentiate between different slates using initial rates of increase in water absorbency and it suggests that the order of durability is I/K-3>> W/F-8 > S-1.

However because the rate of increase in water absorbency is not constant but increases with the degree of weathering predicting the life expectancy of a slate based on an exponential model is more appropriate. This should be done by extrapolating the exponential model to a pre-determined limit, ideally one that is indicative of incipient failure. However this was not possible, even though the procedure was run for nearly a year, there was insufficient time to test any of the slates to the point of observed failure. The Welsh sample W/F-8 showed measurable deterioration after a hundred wetting-and-drying cycles, i.e. there was an increase in the rate of water absorption with time, an increase in variance of the results, as well as a visual change in that brown staining developed. However, there was nothing to suggest that failure was imminent. Hence it was not possible to define a level of water absorption indicative of failure on which to base estimates of the durability of a slate. Instead it was decided to assess the durability of the various slates relative to that of a standard. The standard chosen was the Welsh slate which was assigned a life expectancy of a hundred years, a figure that is often used in trade literature. Although new slates have a water absorption limit of <0.3%, that of used slates which are recyclable have values of <1.0%. Therefore the limit for used slates is set within this range at 0.5%. The W/F-8 regressional curve was extrapolated to determine the number of cycles necessary for the water absorption to reach this limit (Table 4). This was repeated for the other two samples and their life expectancy values were calculated *pro rata*. It was found that the Irish slate has a life expectancy of over 300 years while that of the Spanish slate is 40 – 50 years. To check the effect of the water absorption limit on the results, estimates of durability were recalculated using limits of 0.75% and 1.0%. It was found that, the actual value used within the range 0.3% to 1.0% makes little difference to the relative durability of the slate e.g. higher limits of 0.75% and 1.00% increase the relative durability of I/K-3 to 349 and 357 years respectively. Similarly the values for S-1 are increased slightly to 56 and 58 years respectively.

These results were then compared with the reputation of the slates in question, the Killaloe slate has a reputation of lasting over 200 years and several examples of such longevity was seen on the roofs of early 19$^{th}$ century houses in the area surrounding the Killaloe quarries. The usual figure used in the trade for Welsh slate is 100 years and that for Spanish slate expectancy of 30 years. This demonstrates that the predicted life expectancy values agree with those generally accepted by the roofing industry.

**Table 4 Estimating the life expectancy of a slate relative to a standard.**

| *New slates* | *No of test cycles* | *Equations of regressional curves* | *Increase in thickness after 100 cycles (%)* | *Visual change* | *No. of cycles to 0.5%* | *Life expectancy relative to W/F-8* |
|---|---|---|---|---|---|---|
| I/K-3 | 142 | $y = 0.00007x+0.1195$ | 1.7 | No change | 5436 | 390 |
| | | $y = 0.11956e^{0.0006x}$ | | | 2385 | 333 |
| W/F-8 | 140 | $y = 0.0003x+0.0837$ | 1.7 | Brown stains | 1390 | 100 |
| | | $y = 0.\ 0837e^{0.0025x}$ | | | 715 | 100 |
| S-1 | 97 | $y = 0.0006x+0.1484$ | 3.1 | Brown stains | 587 | 42 |
| | | $y = 0.1496e^{0.0033x}$ | | | 366 | 51 |

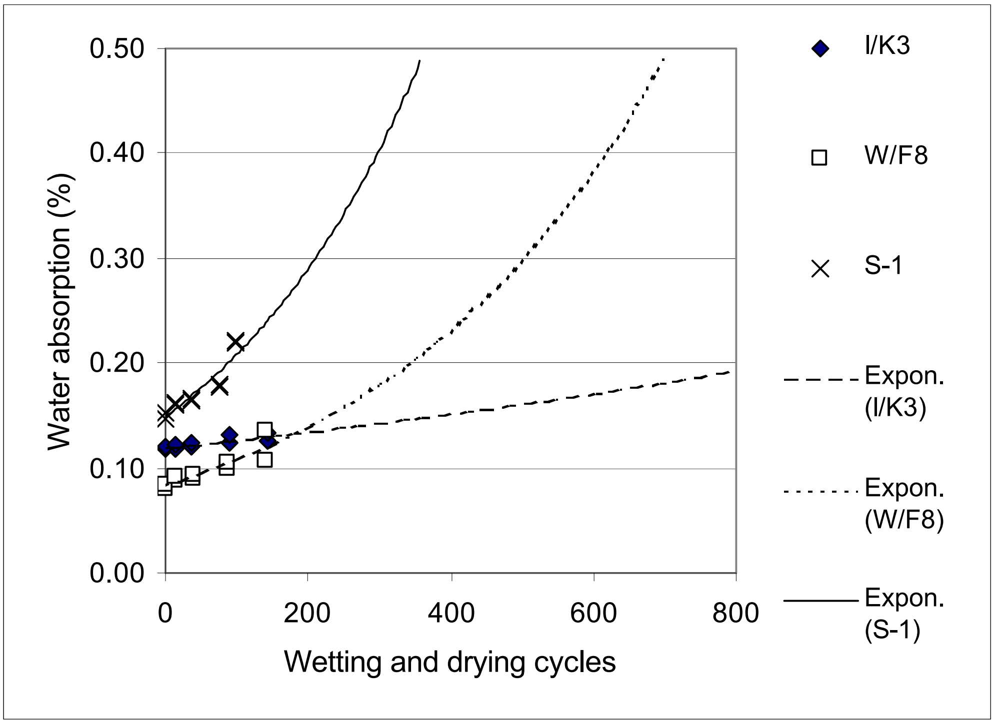



**Fig. 4 Extrapolation of water absorption to a limit of 0.5% using an exponential model**

Most of the above discussion is based on water absorption figures obtained using the BS 680 refluxing method. A parallel set of results was obtained according to the prEN12326 water immersion procedure. If, as reported by Watkins (1934), it takes a month of immersion in water at room temperature for the weight to stabilise, the prEN12326 test is measuring only the *rate* at which the slate absorbs water. If, again according to Watkins, weight stability is achieved by the boiling method in 48 hours, the BS 680 test measures the *total* capacity of the slate to absorb water. Both methods are valid measures of deterioration, but the BS 680 method, gives larger values and earlier signs of significant change and therefore shortens the length of time needed to evaluate a new slate.

Finally using a shortened version of the weathering procedure, extra slates were tested and estimates of their relative durability determined. It was found that the estimates for the Welsh samples were again between 100 to 125 years and those of the Spanish slates were 40-50 years, thus indicating that the results obtained by the above procedure are reproducible. In addition to the extra Welsh and Spanish samples, two Cumbrian slates were tested and found to have very high durability with estimates of over 125 years (Walsh in press). It is worth noting that the Cumbrian slates performed in accordance with their reputation for longevity even in an urban environment. In contrast these slates fail the British Standard Acid Test designed to identify those slates not suitable for use in a polluted atmosphere. However because of their proven track record in an urban environment an exception has always been made for these slates. No fresh Scottish slates were available for testing, however this work has set up a scale with which to assess the quality of a new source when it becomes available

## 5 CALIBRATION OF RESULTS

An attempt was made to relate the increase in water absorption in the experiment which subjected the slates to repeated cycles of wetting-and-drying, to that observed in slates which have experienced natural weathering on roofs.

A tentative calibration has been achieved based on data collected for the BRE study of weathered samples in 1948/49 (Fig. 5). A linear regression (not shown) of the few data points for naturally weathered slate samples from the *old vein* in the Ffestiniog area gave the following relationship between water absorption **y** and years of natural weathering **x.**

$$\mathbf{y = 0.0024x + 0.162}$$

This implies an initial water absorption figure of 0.162% and an annual increase of 0.0024% per year over a hundred year period. The linear regression for the experimental weathering of a new slate from the *old vein*, gave the following relationship between the water absorption **y** and number of cycles of wetting-and-drying **x** (Fig.3b)

$$\mathbf{y = 0.0003x + 0.0837}$$

This gives a rate of increase in water absorption of 0.0003% per cycle. By comparing these two relationships, it is possible to get a rough equivalence between natural and experimental weathering as 8 wetting-and-drying cycles per year. However the rate of increase in water absorption is not linear but increases with time so that a better comparison is based on the following exponential regressional line for natural weathering (Fig.5)

$$y = 0.1673e^{0.0089x}$$

Comparing this with that obtained for experimental weathering (Fig. 4 and Table 4), it is found that it takes 715 cycles to reach a water absorbency value of 0.5% as compared to 125 years of natural weathering so that 5-6 cycles of experimental weathering equates to one year of natural weathering.

An alternative approach was taken by comparing the properties of the new Killaloe slate with those of a used slate from the same quarry which is supposedly 200-year-old. This is a green slate with a gritty texture similar to the new sample. There are some visual signs of weathering such as slight brown staining on both surfaces, the exposed part of the upper surface is blackened and there are some rusty marks around the nail holes. Other changes due to weathering were investigated using XRD and XRF analyses (Walsh 1999).

It was found that water absorption had a value of 0.181%. Assuming that the sample had an initial water absorption similar to that of new slates from the same quarry, this corresponds to an increase of 0.081% or 0.00041%/year. This figure, when compared with the rate of increase of 0.00007% found experimentally in Fig. 3a, equates to 6 cycles of wetting-and-drying being equivalent to one year of natural weathering. However a better equivalence is again based on the exponential model of $\mathbf{y = 0.1038e^{0.0028x}}$. Using this equation it was found that it would take 561 years to reach a water absorption of 0.5%. which when compared with the 2385 cycles necessary to reach the same limit experimentally (Table 4) equates to 4 cycles of wetting-and-drying being equivalent to one year of natural weathering.

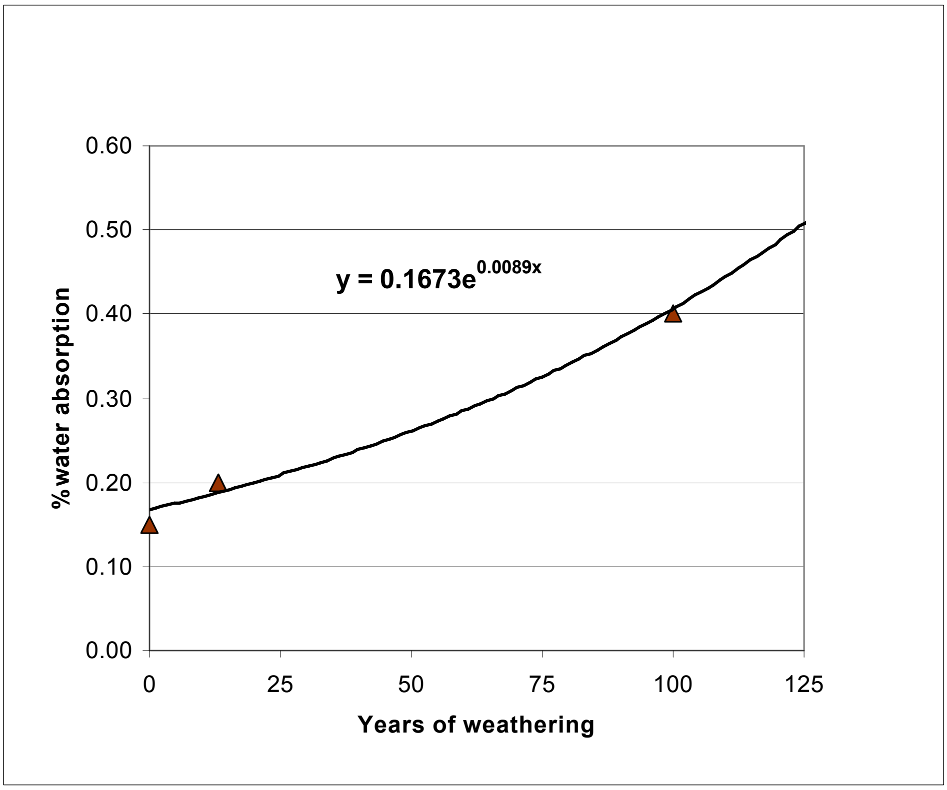



**Fig. 5 Natural weathering of Ffestiniog slate**

Given all the variations possible in the history of the different slates, let alone their imprecise ages (based on information from second-hand slate dealers), it is only possible to say that *4-8 experimental wetting-and-drying cycles are equivalent to one year of natural weathering*. Even the apparent agreement of the two estimates based on I/K-3 and W/F-8 above may be spurious and it is planned to carry our further comparisons between the effects of experimental and natural weathering in a Scottish climate.

Other changes is the properties of a slate due to weathering continue to be investigated, these include changes in the mineralogy and loss of strength. Other climatic environments will also be investigated such as exposure to acid vapour and biological weathering

## 6 CONCLUSIONS

Slates, experimentally weathered using wetting-and-drying cycles, show some of the characteristics of deterioration seen in naturally weathered slates. The increase in water absorption is one of the quantifiable effects which, when compared with that observed in a naturally weathered slate, gives an estimate of the number of cycles of wetting-and-drying equivalent to one year of natural weathering. Although the rate of deterioration increases with time, it is possible to use the **initial rate** at which deterioration takes place as a relative measure of the vulnerability of the slate to weathering. However a better estimate of the relative durability of different slates is based on an exponential model.

To minimise the effect on durability due to differences in the environmental conditions, rates of increase in water absorption were compared relative to that of a standard. The standard chosen was a Welsh slate to which was assigned a life expectancy of 100 years. All the other Welsh slates tested were found to have similar life expectancy values, the Cumbrian slates were slightly higher and the Spanish slates were approximately half that of the standard. These results agree with estimated values found in the trade literature (Walsh in press).

Being able to estimate the relative durability of different slates is the first step in determining the whole life costing of roofing material. A slate with a life span of approximately 50 years would have to be replaced several times within the useful life of one lasting 200 years. As a result, the less durable material would incur higher energy input overall from the extraction of the basic material at source to the transport and maintenance costs of the finished product when compared to its more durable counterpart. To date assessing the quality of different slates is reliant on the reputation of the slate in question. But reputation based as it is on past performance is not always the best guideline as there are many reasons why the quality of a slate from a particular source may change. This research attempts to set in place a protocol for assessing the quality of a slate objectively, enabling the identification of those slates that are easily weathered.

## 7 ACKNOWLEDGEMENTS

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# Computer 3D Microstructure Modeling And Prediction Of Hydration And Strength Of Lunar-like Cement With Steam

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**Summary: This paper presents an attempt to create a computer based cement hydration model for three types of cements, one conventional and two lunar type cements. Here, lunar cements are named like that as their compounds are very similar to the candidate lunar materials.Using such simulants is efficient in order to simulate and predict lunar cement properties and behavior. Computer simulation is based on that developed by NIST 3D hydration and microstructural model. Four different models are established for the lunar and portland cements. Models represent two wet mixtures with lunar cement, one is with addition of gypsum and both lunar cement and conventional portland cement threaten under dry-mix / steam injection (DMSI) method. In order to simulate the dry-mix / steam injection hydration process, the algorithm in the 3D computer simulation program is modified respectively with the different nature of steam hydration, hydrates formed, their amount and formation in time line. Based on the derived simulation models, the hydration process, morphological and chemical compositions of the hydrates formed, percolation properties of the different mixes are compared. An evaluation of the predicted properties and experimentally determined is carried out as well. The results show that this type of computer modeling and behavior prediction could be very useful in order to plan and evaluate costly and unique (like in case with lunar cement) experiments.**

**Keywords: Microstructure modeling; hydration; strenght development; computer image-based model; percolation.**

## 1 INTRODUCTION

In the present studies on materials which could be produced in outer space and the moon, concrete is considered as one of the suitable materials. There are studies regarding possibilities to produce cements from lunar materials and materials which chemical compounds are very closed to lunar one (Horiguchi et al. 1996). Actually many constrains exist if the experiments with real lunar materials are to be conducted. Main are the availability of the real lunar materials and if substitutes are used, how close to the real one they are. It will be very helpful if computer-based models of microstructure of such cements, hydration process and transport properties could be created and used (Bentz 1997).

The major work in computational materials science of cement and concrete and computer-based models dates from the last 10 to 15 years and is powered by the advancements and progress made in the computer processing speed and memory capacity. Cement and concrete as a random materials are a very suitable object to be characterized, described and predicted in its properties by applying computer-based models. Such models are developed for describing interfacial transition zone (ITZ), microstructure in concrete; leaching of (calcium hydrates) CH from microstructure; set point determination; hydration rates, heat of hydration, and chemical shrinkage; effects of cement particle size distribution (PSD) on cement and concrete properties (Bentz 2000). The development and use of similar computer image-based models will highlight the microstructure characteristics of lunar like (high aluminates) cements.

Another important point to consider is that dry mix/steam injection method is belived to be applicable for moon or outer space environment. But the hydration process develops in this type of system diferently. It is important to create a computer based model for the hydration and microstructure in dry mixes subjected to steam injection.

## 2 OBJECTIVE

The objectives of this study are: to reconstruct 3D digital microstructure of two high aluminate cements, one of them with a gypsum addition and one ordinary portland cement, which are tested in two ways: conventional (wet mixtures) and DMSI (dry mixtures); to estimate the hydration rate and compressive strength based on the established models; to compare predicted results with the measured one from previous experiments.

## 3 BACKGROUND

Both the computer image based models and dry mix/steam injection method are relatively new technologies which developments aim to highlight the microstructure of existing and future materials with enhanced behaviour. The results of previous studies (Lin et al. 1998) showed that DMSI causes rapid hydration and very high strength compare to wet mix method, which is a strong base for further research.

## 4 COMPUTER IMAGE-BASED MODEL

### 4.1 Testing Procedure

Here, experimental data from two different sources have been used (Horiguchi et al. 1996; Lin et al. 2000) in order to construct computer image-based model representing the microstructure before and after hydration. The details for the mixtures used are given in the Table 1 below:

**Table 1. Mixtures names, cement fineness and types, curing conditions and compressive strength.**

| Type Cement | Mixture | Fineness $m^2/kg$ | W / C | Curing | SEM Image | CS –24h (MPa) | CS –3/7 d. (MPa) |
|---|---|---|---|---|---|---|---|
| Lunar | WM1 | 357 | 0.835 | Wet | CD357 | N.A. | 8.4/12.2 |
| Lunar+Gyp | WM2 | 357 | 0.600 | Wet | CD357 | N.A. | 1.9/2.9 |
| Lunar | DMS | 357 | 0.2 | Dry | CD357 | 21.9 | N.A. |
| Port | DMO | 387 | 0.2 | Dry | CU387 | 38.8 | N.A. |

According to the reference source (Horiguchi et al. 1996), for all different cements, a series of 4 cm mortar cubes were prepared. The standard Japanese mix procedure is applied where the weighed cement, sand and water (as shown in Table 2) were mixed in a electric powered mixer for 4 minutes. The fresh mortar then was cast in 4x4x16 cm prismatic molds. After compacting and finishing, the mortar test specimens were cured at 20 degree C and 60% humidity for wet mix method, and into autoclave unit for steam curing at 170 C and pressure of 7 $kgf/cm^2$ for 24 hours. The temperature regime is presented on Fig. 1.

**Table 2. Mix Proportions for Wet Mixes (Horiguchi et al. 1996) and Dry Mixes.**

| Code | Water, cc | Cement, g | Sand, g | W/C |
|---|---|---|---|---|
| WM1 | 334 | 400 | 800 | 0.835 |
| WM2 | 240 | 400 | 800 | 0.600 |
| DMS | 0 | 400 | 800 | 0 |
| DMO | 0 | 400 | 800 | 0 |

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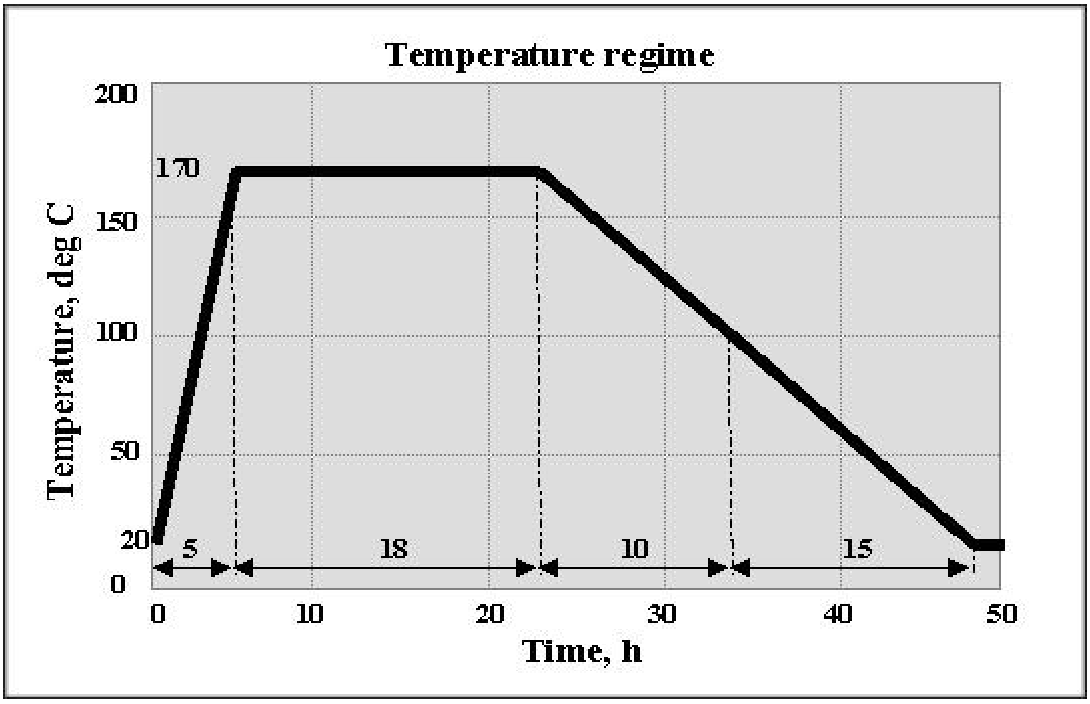



**Figure 1. Temperature regimes for dry mix / steam injection.**

After the curing, the test specimens were tested to determine the hydration products (by X-ray diffraction) and scanning electronic microscopic analysis for highlighting the morphological features of hydration products. Also examined are the degree of hydration and compressive strength.

### 4.2 3D Digital reconstruction of microstructure

The reconstruction and 3D microstructure generation and hydration simulation proces'flow diagram is presented on Fig. 2.

#### 4.2.1 Generation of 3D image and distribution of phases

According to the procedure for generation of 3D microstructure of cement and concrete (Bentz 2000), a scanning electron microscope image and particle size distribution are the initial inputs. Particle size distribution is on a number basis, and the phase volume fraction and phase area fraction are obtained based on digital SEM images. Then by applying the developed programs for reconstruction of initial 3D microstructure (Bentz 2000), its first approximation is received. Further, the distribution of phases based on phase volume and phase area fractions is performed by applying NIST procedure. Figure 3(a, b, c, d) presents the data input, images,

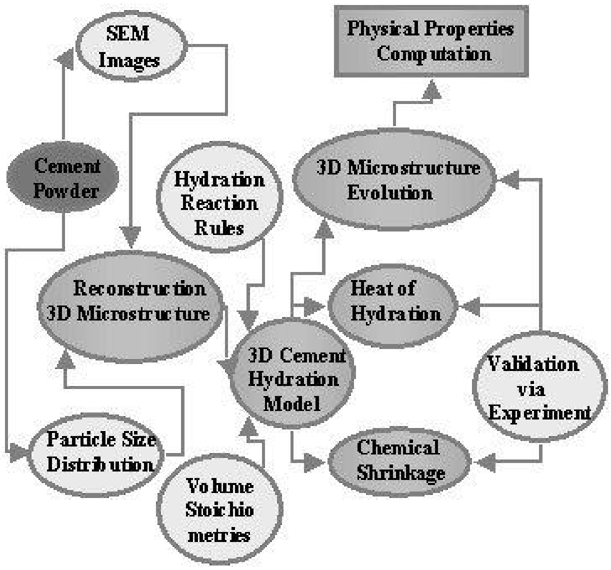
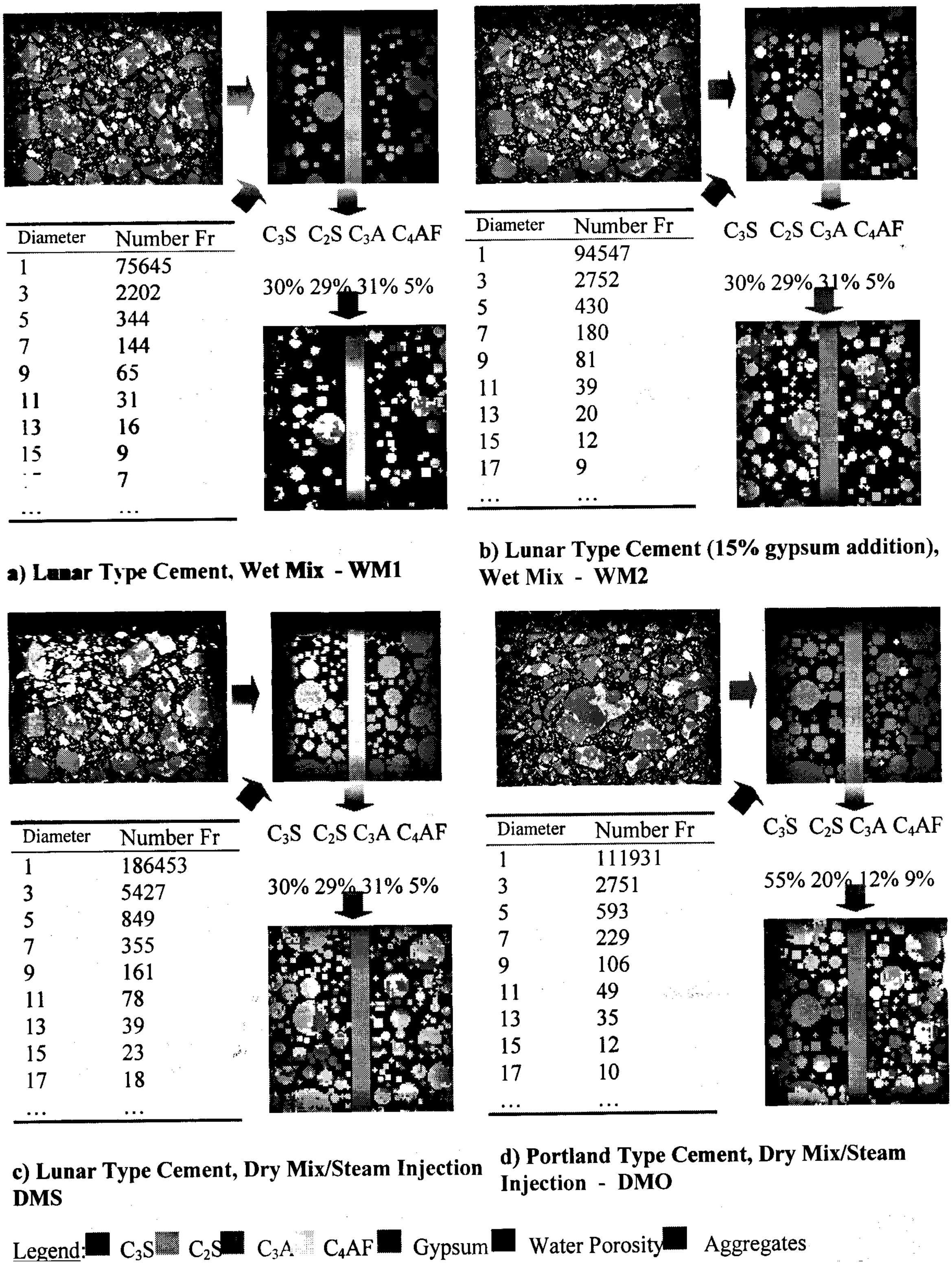



**Figure 2. Modeling and experimental flow for hydration simulation.**




| Diameter | Number Fr |
|---|---|
| 1 | 75645 |
| 3 | 2202 |
| 5 | 344 |
| 7 | 144 |
| 9 | 65 |
| 11 | 31 |
| 13 | 16 |
| 15 | 9 |
| [illegible] | 7 |
| ... | ... |

**a) Lunar Type Cement, Wet Mix - WM1**

| Diameter | Number Fr |
|---|---|
| 1 | 94547 |
| 3 | 2752 |
| 5 | 430 |
| 7 | 180 |
| 9 | 81 |
| 11 | 39 |
| 13 | 20 |
| 15 | 12 |
| 17 | 9 |
| ... | ... |

**b) Lunar Type Cement (15% gypsum addition), Wet Mix - WM2**

| Diameter | Number Fr |
|---|---|
| 1 | 186453 |
| 3 | 5427 |
| 5 | 849 |
| 7 | 355 |
| 9 | 161 |
| 11 | 78 |
| 13 | 39 |
| 15 | 23 |
| 17 | 18 |
| ... | ... |

**c) Lunar Type Cement, Dry Mix/Steam Injection DMS**

| Diameter | Number Fr |
|---|---|
| 1 | 111931 |
| 3 | 2751 |
| 5 | 593 |
| 7 | 229 |
| 9 | 106 |
| 11 | 49 |
| 13 | 35 |
| 15 | 12 |
| 17 | 10 |
| ... | ... |

**d) Portland Type Cement, Dry Mix/Steam Injection - DMO**

Legend: C$_3$S C$_2$S C$_3$A C$_4$AF Gypsum Water Porosity Aggregates

**Figure 3. SEM Images, PSD, 2D slices of 3D initial digital mono microstructure and the same after the distribution of the phases for the lunar and portland cements and for wet &dry mixtures.**

PSD tables and the initial 2D slices of 3D mono C$_3$S microstructure and all phases'microstructure after the distribution. The assumptions are: the resolution is 1 μm/pixel; particles are placed from largest to smallest; gypsum (volume fraction from the total) and cement follow the same particle size distribution.

### 4.2.2 Hydration reaction rules

In the present version of a 3D cement hydration and microstructure development modeling package CEMHYD3D (Bentz 2000), the hydration simulation is based on phases, reactions to be considered in the hydration and the rules for the dissolution of solid material, the diffusion of the generated diffusing species, and the reactions of diffusing species with each other and with solid phases. In the reconstructing of the wet mixtures microstructure (WM1 and WM2), the same assumptions for the phases, reactions and the rules in the cellular-automata are applied. But for the reconstruction of dry mixtures some modifications are proposed and applied.

Reactions and rules applied to wet mix cement test specimens. The hydration of wet cement mixtures is well studied by many scientists (Taylor 1990; Mindess and Young 1981) and consists of dormant, acceleration (including setting) and deceleration periods. Through the dormant period not significant changes occur, through the acceleration period most of calcium silicate hydrate (CSH) and calcium hydroxide (CH) gel is formed and in the deceleration period, tricalcium aluminate (C3A) and tetracalcium aluminoferrite (C4AF) are surrounded by CSH and gradually become stronger. The cement model reactions (Bentz 1997) included in NIST model are:

a) Silicate reactions

$C_3S + 5.3H \rightarrow C_{1.7}SH_4 + 1.3CH$ $\qquad$ $C_2S + 4.3H \rightarrow C_{1.7}SH_4 + 0.3CH$

b) Aluminate and ferrite reactions

$C_3A + 6H \rightarrow C_3AH_6$ $\qquad$ $C_4AF + 3CSH_2 + 30H \rightarrow C_6AS_3H_{32} + CH + FH_3$

$2C_3A + C_6AS_3H_{32} + 4H \rightarrow 3C_4ASH_{12}$ $\qquad$ $2C_4AF + C_6AS_3H_3 + 12H \rightarrow 2C_4ASH_{12} + 2CH + 2FH_3$

$C_3A + 3CSH_2 + 26H \rightarrow C_6ASH_{32}$ $\qquad$ $C_4AF + 10H \rightarrow C_3AH_6 + CH + FH_3$

These reactions are implemented in cellular automata rules for the dissolution of solid material step, the diffusion of the generated diffusing species step, and the reactions of diffusing species with each other and with solid phases step.

Reactions and rules applied to dry mix/steam injected cement test specimens. The hydration of dry mix/steam injected cement mixtures is not so well studied and analysed and its research continues. Though some studies provide ideas on the reactions and hydration under steam treatment (Horiguchi et al. 1996; Lin et al. 1998; Lin et al. 2000).

According to them, the steam condenses and forms a moist coating on the cement particle surface. The moist coating thickness increases with the increase of steam condense as a result of capillary pressure. As a result the cement compounds along the contact area start to dissolve and later to form calcium silicate hydrate (CSH) gel. Main difference is that the moisture coating is not sufficient to dissolve calcium oxide in the cement compounds in order to form calcium hydroxide (CH) crystals, which are the first to be formed in wet mix cement.

The reactions to be considered then in the model are:

a) Silicate reactions

$2C_3S + 3H \rightarrow C_5S_2H_2 + CH$

$C_2S + 1.17H \rightarrow C_{1.7}SH_{0.87} + 0.3CH$

b) Aluminate and ferrite reactions

$C_3A + 5.56H \rightarrow C_3AH_{5.56}$

$C_4AF + 6.33H \rightarrow C_{3.4}(A,F)H_{5.73} + 0.6CH$

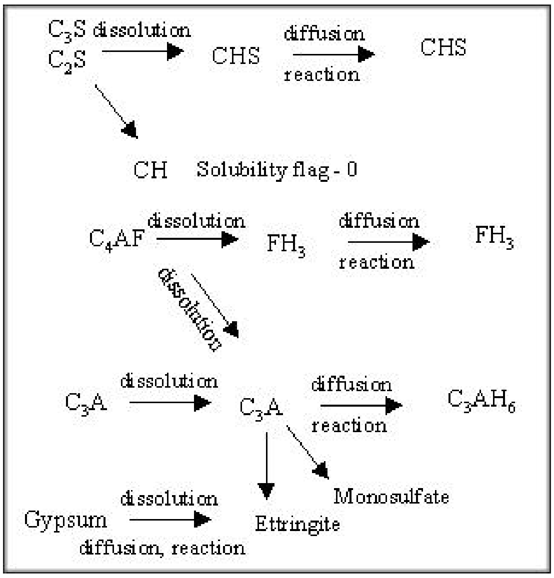



**Figure 4. Dissolution, diffusion and reaction scheme.**

Then the proposed modified scheme for the dissolution, diffusion and reaction steps is seen

In Fig. 4. Compared to the model presently included in NIST, here the calcium hydroxide is consider initially to be not soluble, and the solubility flag is 0 for CH pixels. Another difference is that the dissolution, diffusion and reaction of $C_3S$ and $C_3A$ occur with accelerate speed as a result of the steam pressure in the very beginning of the hydration process.

## 5 RESULTS AND DISSCUSION

After execution of the NIST 3D model and the modified one, the 2D slices of 3D hydrated microstructure is presented in Fig. 5, then the hydration degree, compressive strength and percolation of solids versus hydration are estimated and plotted on Fig. 6 and Fig.7. While simulating the steam hydration the defined temperature regime is applied (described in Fig.1), and the activation energy is converted to the one, which corresponds to the curing conditions. The solubility flag of CH pixels is set to 0 as well.

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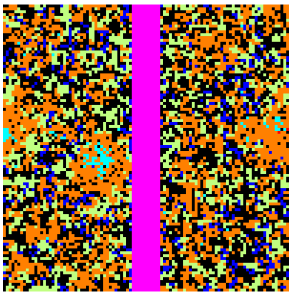
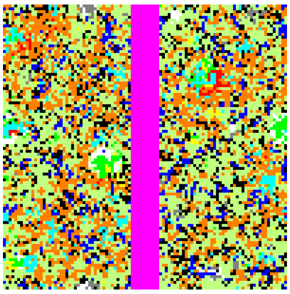
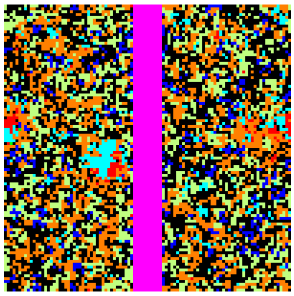


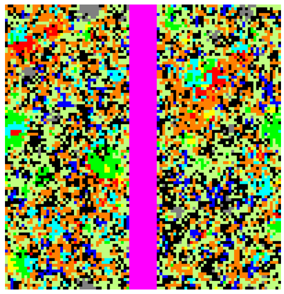
**a)WM1hydrates at 2000 cycles**



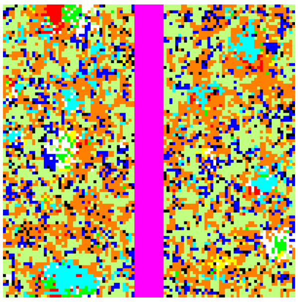
**b)WM2 hydrates at 2000 cycles**



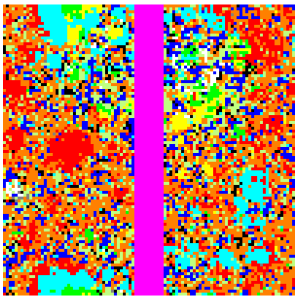
**c)WM1hydrates at 550 cycles**



**d)WM1hydrates at 540 cycles**



**e)DMS hydrates at 2000cycles**



**f)DMO hydrates at 2000cycles**

Legend:

| | |
|---|---|
| $C_3S$ | CHS |
| $C_2S$ | CH |
| $C_3A$ | Aggregates |
| $C_4AF$ | Other Hydrates |
| Gypsum | Empty Porosity |
| Water Porosity | |

**Figure 5. 2D slices of 3D hydrated microstructures.**

The results on Fig.5 give an idea for how the microstructure has changed with the cycles elapsed from the start of the hydration. As the WM1 mixture is with very high W / C, the water porosity is still present and not reach by solid phases even at 2000 hydration cycles. The WM2 mixture shows different hydration development, as a result both of W / C and presence of gypsum, which continues to react with $C_3A$ until is entirely consumed. The hydration occurs faster than in case of WM1 mixture and at 2000 cycles already an empty porosity is observed. On the other hand, steam injected and dry mixtures as a result of almost zero W / C show very intensive hydration in the early stage, proved by the graphics on the right side of Fig.6. The 2D slices of the 2000 cycles hydrated DMS and DMO microstructures also show the distribution of hydrated products, and an empty porosity in DMS case can be observed. It could be explained with the fact that CH is formed in less amount than compare to wet mixtures.

When compare the compressive strength of both predicted (based on the model) and experimentally measured (Table 1), good agreement is found in case of WM1 and DMO mixtures but significant discrepancy in case of WM2 mixture and slight in DMS case. Two reasons could be indicated: the high volume presence of gypsum in WM2 mixture, and the approximate SEM input images used.

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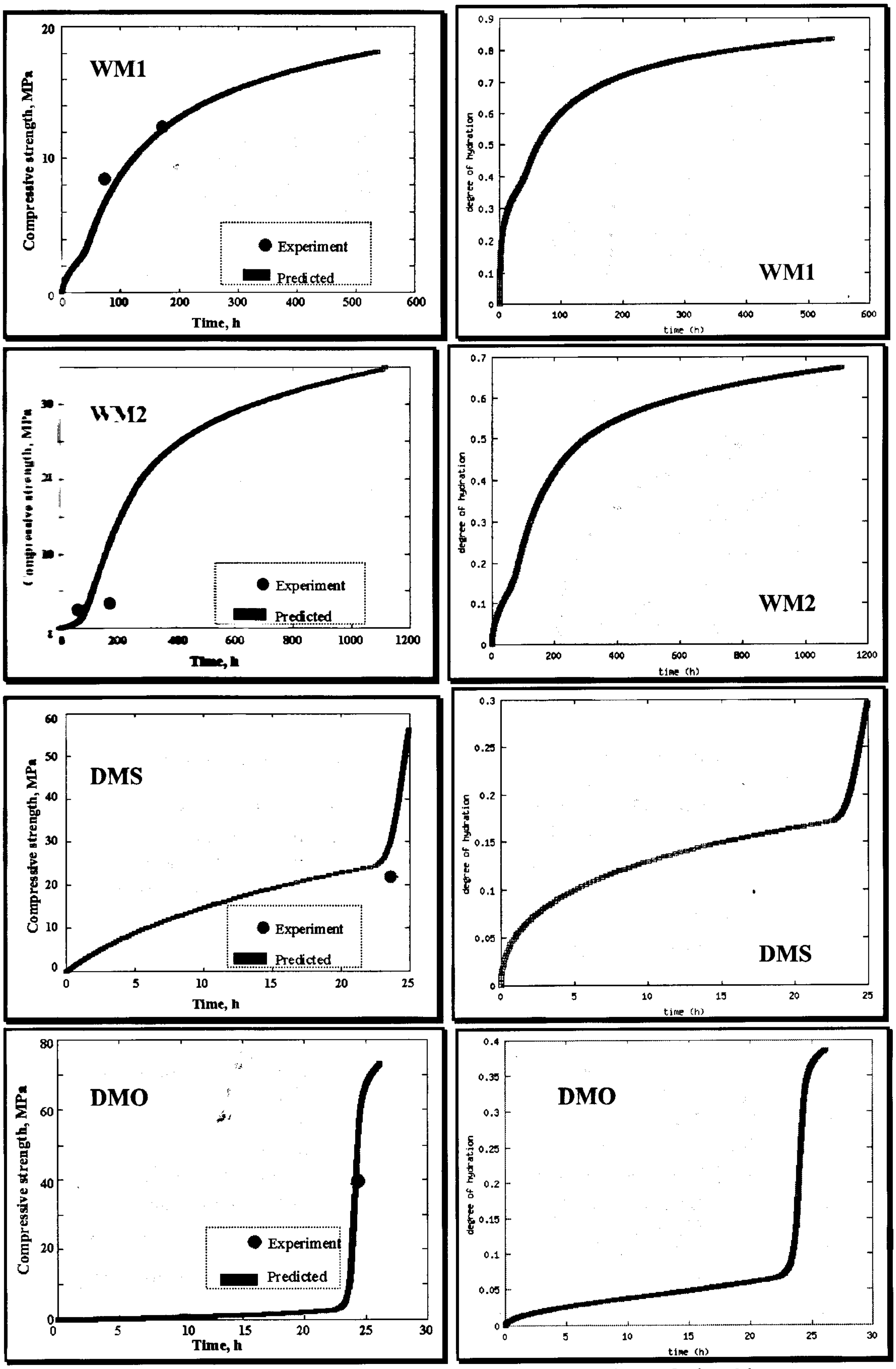


**Figure 6. Compressive strength-time and hydration-time relationships.**

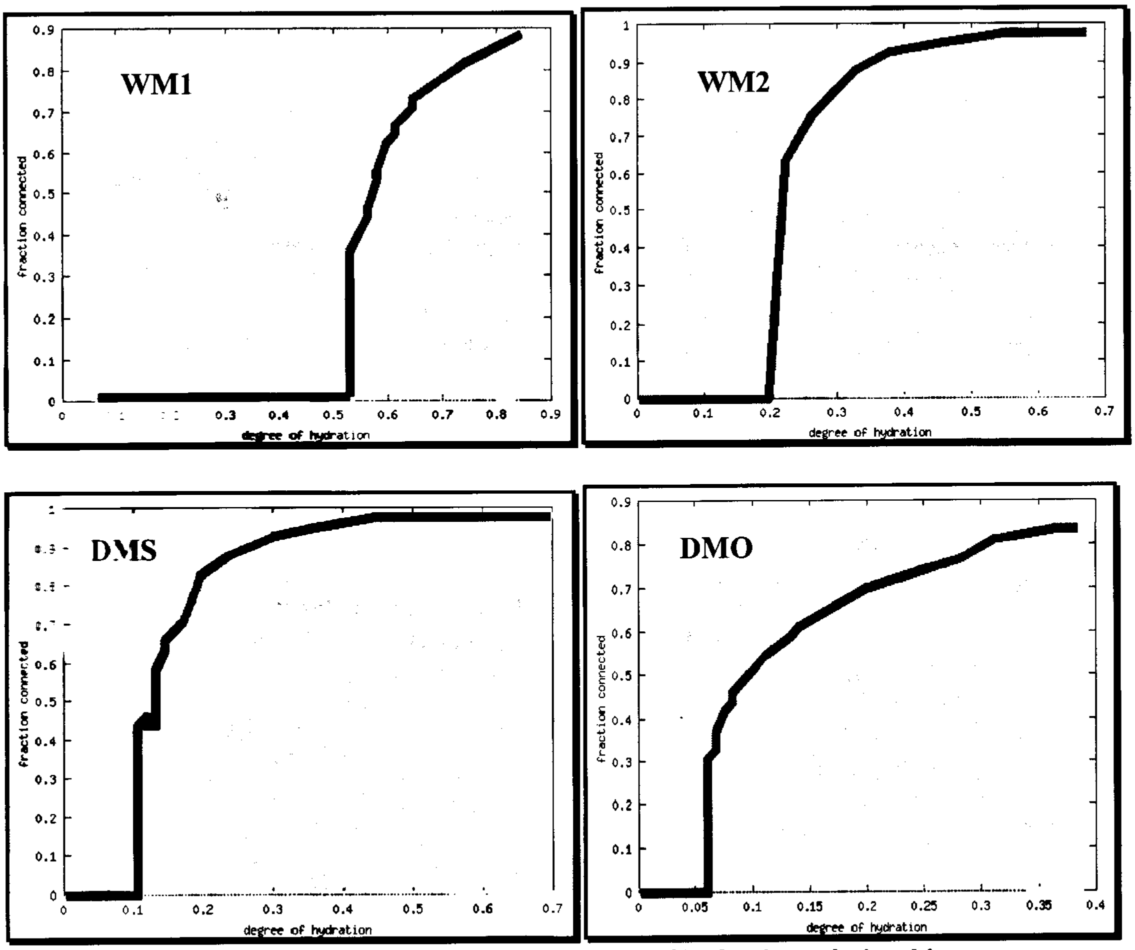


**Figure 7. Percolation of solids – degree of hydration relationship.**

The solid percolation and degree of hydration relationships for the four studied cases reveal that hydrates are formed on quite early stage when steam injection is applied. In case of large W / C (WM1) fractions get connected only at 0.5 hydration degree, and with the decrease of W / C – much earlier: 0.2 hydration degree in case of WM2, and about 0.1 in DMS/DMO case.

## 6 CONCLUSIONS

Based on the simulation, the models established, and the comparison with some experimental results, it could be summarized that computer image based models are good tool to predict physical and transport behaviour of cements and concrete. The simulation results show that W / is significant factor for developing of hydration. Predicted and experimentally measured compressive strengths showed relatively good agreement with exception of the case of WM2 mixture. It is of great importance to have an accurate characterization of the starting materials in order to develop a realistic hydration model. It is necessary to conduct more experimental tests and simultaneously develop realistic describing them models.

## 7 ACKNOWLEDGMENTS

The authors would like to express their sincere gratitude to the researchers of Building Materials Division, Building and Fire Research Laboratory, National Institute of Standards and Technology, USA and authors of the electronic monograph: “Modeling and measuring the structure and properties of cement-based materials”, Dr. Dale Bentz, Dr. Edward. Garboczi, et al. for kindly providing the source of CEMHYD3D: A Three-Dimensional Cement Hydration and Microstructure Development Modeling Package. Version 2.0 and the supporting materials.

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# Toward Practical Application Of Factor Method For Estimating Service Life Of Building

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**Summary**: **Estimation of service life is a prominent important phase in the process of the service life planning. Among various available estimation methods for a whole building and building components, the Factor Method, modification of reference service life by factors to take into account of the specific in use conditions defined in ISO (ISO 15686-1), would be one of the realistic and practical method at present stage until the another alternative methods would be improved.**

**In view of the fact that relatively small amount of information has been reported on the principle of this method and also few works conducted pertaining to the applied results of this method, this paper investigates into the necessary performance data to establish the reference service life, the way and measures for preparation or classification of each factor, and it also shows some details that have been worked out in the performance based service planning system in Japan. The paper also points out that further investigation of the factor method could contribute to quantify and categorize the relevant items taken into consideration of service life, that would be foreseeable and unavoidable process in any other prediction system, and the factor method could be expected to apply for the stock management system as the estimation of residual service life of existing assets as well as their maintenance planning.**

**Keywords. Service life planning, Service life estimation, Factor method, Durability, Degradation**

## 1 INTRODUCTION

At the beginning, to ensure that key terms in this paper are understood, the following are presented;

Durability: Capability of a building or its parts to perform its required function over a specified period of time under the influence of the agents anticipated in service (ISO 15686-1). <Ability of a building, its parts, components and materials to resist the action of degrading agents over a period of time AIJ (1988) is commonly accepted in Japan>

Service life: Period of time after installation during which a building or its parts meets or exceeds the performance requirements (ISO 15686-1)

Service life planning: Preparation of the brief and design for the building and its parts to achieve the desired life, for example in order to reduce the costs of building ownership and facilitate maintenance and refurbishment (ISO 15686-1). <Period of time during which all essential performance characteristics of a properly-maintained item (product, component, assembly or construction) in service exceeds the minimum acceptable values, Frohnsdorff & Nireki (1944) >

Design life: Service life intended by the designer (ISO 15686-1). <Service life that the designer intends an item (product, component, assembly or construction) to achieve conditions and maintained according to a prescribed maintenance management plan, Frohnsdorff & Nireki (1944) >

In practice, the service life planning is still not easy work for designers if they could understand the concept of it and wishes to introduce it into their building design. Though information on durability and degradation of building components or materials are provided in scientific documents for limited objectives, the necessary information for service life planning is rather hard to extract from those documents and usually they do not include a clear guide for the designer.

To cope with this situation, various domestic and international systematic approaches to service life have been made, and in fact interest in the possibility of predicting the service life of building and/or components has grown, as has the interest of designers in the necessity of providing service life planning.

These interests would assuredly foster progress towards making reliable service life prediction systems and towards the goal of providing service life planning as performance-oriented design system – setting a target (design life) at the first stage, then detailed design, work execution and maintenance planning follow to achieve it.

## 2 DURABILITY AS PROMINENT PERFORMANCE OF BUILDING

Durability is an important factor that could not be ignored when considering the performance of buildings. Durability has previously been stressed by the requirements, effective use of natural resources and saving energy, however, these requirements have turned to more wider and critical ones due to the recent social needs or requirements to the construction industry; not mere requirement for each building or a project but as nation-wide and global requirements such as saving energy at production of building materials leads to preservative use of global resource, to reduction of environmental loads then prevention global warming.

## 3 SERVICE LIFE PLANNING

The service life of buildings is influenced by the conditions throughout all the stages from planning, design, contract, construction, running, maintenance to demolition. Thus, the effectiveness of service life planning cannot be assured unless all conditions taken into account at design stage would be realized throughout whole life of a building from planning to demolition. The other prominent factor enable to realize the service life planning would be full understanding of this design system among all the persons concerned with a building in question as such planner, architect, designer, owner, property manager, caretaker, auditor, constructor, product manufacturer, user and occupant. In other words, the service life planning could be accomplished on the basis of mutual agreement among all sectors concerned and also the design life would be an indicator of consensus.

From the viewpoint of practical access to take service life or durability into the design of a certain building project, there are various so-called guides or rather huge amount of academic and technical documents for detailed technical issues, however very few of them are applicable for design of as a whole building or a part of it.

### 3.1 Service Life planning system

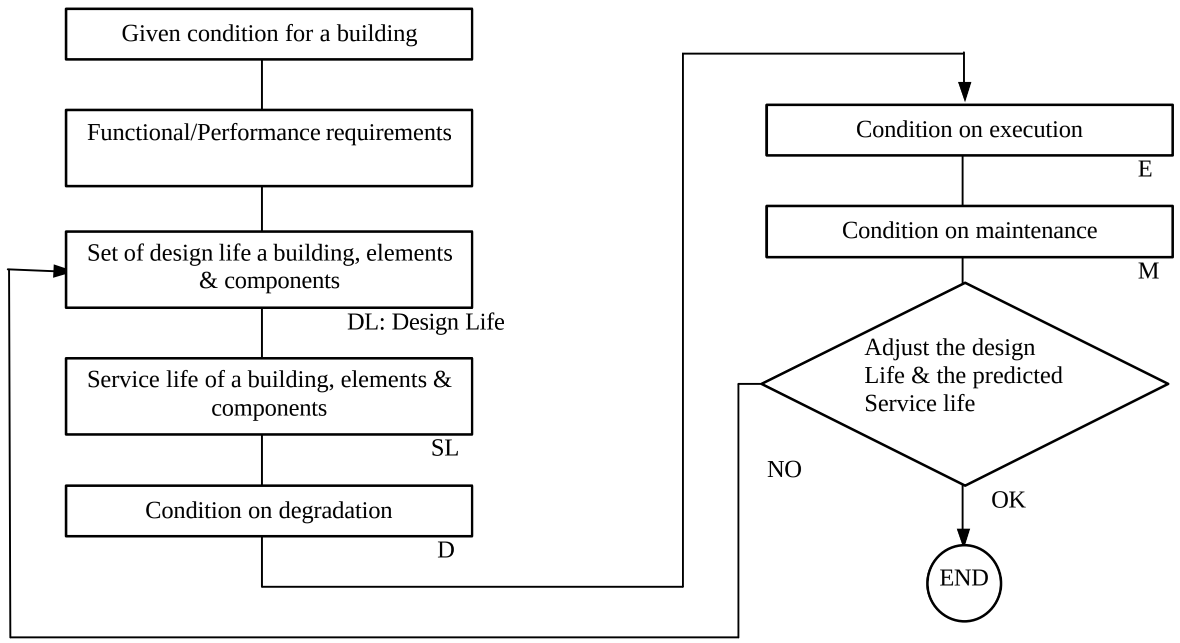
The followings are some typical approaches that intend to improve the service life planning for buildings and some of them are already standerdised.

#### 3.1.1 Ministral R & D for service life planning in Japan

Along with the interest of requirement on durability, the Ministry of Construction (MOC, Ministry of Land, Infrastructure & Transport, MLIT of present) started a five year R & D “Development of Techniques for Improving Life Time of Building. (Fiscal year 1980 – 1984)” This large scaled joint governmental research institute, university and relevant industry, total amount of member for technical committees and sub-committees reached to some 650 within five years, was promoted by Building Research Institute (BRI) and the research target could be summarized as below. Nireki et. al (1983)

| | | | |
|---|---|---|---|
| 1 | Maintenance for existing building<br>Degradation agent & Diagnosis<br>Repair & Upgrading | 2 | Improving service life of new building<br>Requirements, design includes design criteria Higher-durability product - Structural - Non load bearing - Services<br>Site works & Management |
| 3 | Evaluation method<br>Maintenance technology<br>LCC / LCA<br>Maintenance & management | 4 | Improving service life<br>Guide to Service Life planning<br>Guide to Execution & site work<br>Guide to Maintenance & management |

The guide for service life planning are proposed based on the research results obtained from the elemental projects? to? as in above, and the basic planning concept throughout all objectives could be outlined as shown in Figure 1. Nireki (1995)




$SL \fallingdotseq SS \times D \times E \times M$

SS: under normal use condition (equivalent to Reference Service Life in ISO 15686-1)

**Figure 1. Concept of service life planning in the Ministral R & D project**

The following guide to service life planning had been established.

- Reinforced concrete building
- Piping and plumbing
- Steel framed building
- Mechanical & electrical installations and equipments
- Wooden building
- Surface coating for external wall
- Masonry coating for external wall
- External wall tiling
- Cement mortar finishing for external wall
- Aluminum external cladding and window and door set
- Waterproofing for flat and gradient roof
- Sealing

In this concept, to estimate the service life of a whole or part of building, building element, component etc. should be one of the key process, then, a rather simple calculation method, service life could be obtained by multiplication of several conditions (factors) that would effect the service life of the objective in question, had firstly been introduced.

3.1.2 AIJ guide for service life planning in Japan

Architectural Institute of Japan (AIJ) decided to organize a technical committee on durability (Chairperson: Prof. Shirayama, Core member: T.Nireki, M.Sato) in 1979, aiming to systematize the concept of durability in the field of building engineering.

In 1989 the committee compiled “Principal Guide for Service Life Planning of Building” (referred to as AIJ Guide (Japanese) ) which was on the basis of the results of committee works for some nine years and referring the results from the Ministral R &

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D project as describe above. The principle of service life planning in this guide was fundamentally on the almost same track as in the Ministral R & D project (1980 - 1984), as shown in Figure 1, and some service life estimation methods by applying the multiplication method for concrete, steel, wooden component etc. were compiled in the Appendix of the guide.

In 1993 AIJ Guide (Japanese) was translated into English as, "The English edition of Service Life Planning of Building (referred to as AIJ Guide (English)"and issued from the AIJ. After issuing the AIJ guides, the committee was reformed as a technical committee "Service life planning" (Chairperson Nireki) and active works have been done. Main works stressed in the committee were to make AIJ Guides much more practical taken the recent requirements and technical development into account and also to support the standardization of ISO 15686 series as one of the domestic group in Japan, then, the committee drafted "Service Life Planning for Building, Components and Materials" in 2000.

### 3.1.3 Service life planning in British and Canadian Standard

Both British standard, BS 7543:1992 "Guide to Durability of building and elements, products and components" (BS 7543:1992) and Canadian standard, S478-95 "Guideline on Durability in Building" (S478-95: 1995) are very useful technical documents for design for durability and/or service life planning.

Main notable difference among the Ministral R & D, AIJ Guide in Japan and these standard could be said that the first one is result of authorized research result and the second one is a proposal from the professional body (AIJ), therefore, the both do not still have any regulatory or compulsive character for building design in Japan expect some part of them had or would be specified in the building code or regulation.

### 3.1.4 Service life planning in ISO 15686

Standardizing works for service life planning in the ISO TC59/SC13/WG9 (TC59/SC14 at present) started in 1993 in Japan at just before the opening of 6th DBMC held in Japan. Since the, Part 1: "general principle" (ISO 15686-1) was issued in 2000 and Part 2 (ISO 15686-2): "Service life prediction procedures" was successively issued in 2001, and reminder parts as audit, LCA etc. are under preparation.

The target and the role of Part 1 can be found in the scope of it; this part of ISO 15686 describes the principles and procedures that apply to design when planning the service life of buildings and constructed assets. The systematic process of service life planning is shown in Figure 3 in the standard (ISO 15686-1) and this could be summarized as in Figure 2.

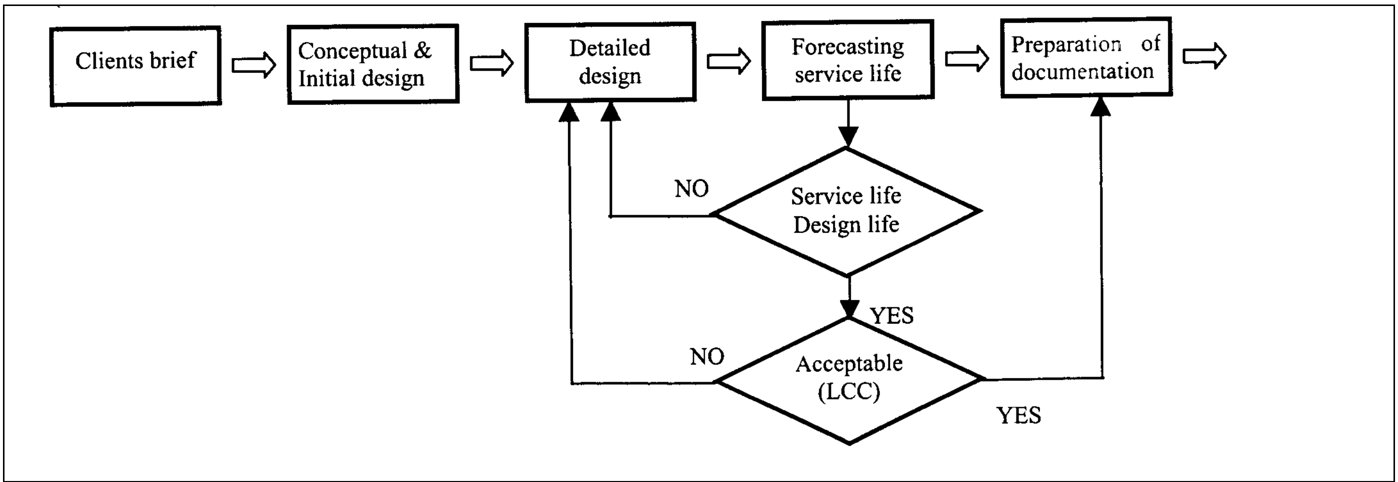


**Figure 2. Stepwise service life planning process in ISO 15686-1**

### 3.1.5 Service life planning in the new code in Japan

At present rather complex work is forced to designer, their burden and risk could be reduced if a certain design life would be selectively provided and that the corresponding components, materials and some details of the service life would be prepared. Above way of service planning has become to be possible for designer including supplier of dwelling system in Japan in accordance with the newly enforced "Housing Performance Indication System" HQAL (2000) in the "Housing Quality Assurance Law" enforced in 2000. HQAL (2000) This performance indication system is intended to provide customers with reliable information on nine types of performance items and their levels.

At present the system is not yet the compulsive measure, therefore, whether to use it or not is left to the designer's, supplier's and customer's choice, however, rather higher ratio of newly built dwelling systems would be expected to apply the system. The nine types of performance selected in the Performance indication system are:

- Structural safety

- Easiness of maintenance
- Lighting & visual environment
- Fire safety
- Thermal environment
- Acoustic environment
- Preventive measures for degradation
- Indoor air quality
- Care for aged occupant

Preventive measures for degradation <direct translation into English> is almost equivalent to durability in English but a slight different meaning due to historial background.

For enforcement of the indication system, "Housing Performance Evaluation Standard" (HQAL 2000) has also been provided for the evaluation and ranking of each performance item and sub-items. In the evaluation standard, preventive measures for degradation, objective is limited to structural part or member of dwelling system, is divided into following three ranks based on the service life of dwelling in question.

Rank 3 : Necessary preventive measures for degradation are provided for some three generations*

Rank 2 : Necessary preventive measures for degradation are provided for some two generations*

Rank 1 : Necessary preventive measures for degradation are provided to conform to the Building

Standard Law.

*One generation term is supposed to be 25 to 30 years. This term corresponds to average duration which one generation would occupy a dwelling in Japan. Therefore, the Rank 3 means the service life of some 75 to 90 years.

The "preventive measures for degradation" is evaluated on the basis of the following conditions. Rank of the "preventive measures for degradation" express the level of preventive measures to sustain the service life of dwelling by the limit state. The limit state is described as the degraded state when the dwellings would reach to any one of the states as in below.

a) The state when the performance or function degrades beyond the allowable threshold, and when it is impossible to recover this degraded state to the allowable limit by means of ordinary repair or partial replacement or renewal.

b) For the case when the performance or function could be recovered back to the allowable limit by repair, however, the economical disadvantage caused by the successive use of the building in question is expected.

In practice, designers or suppliers can select any one of rank out of three ranks depending on their design policy and tactics, according to the evaluation system based on the factor method Motohashi & Nireki (2002), especially for steel and timber dwelling and this selection of a rank would be very similar process to set "design life" in the service life planning of building as already introduced in the Ministral R & D, AIJ Guide even in ISO 15686-1.

## 4 ESTIMATION OF SERVICE LIFI

The following key terms in this paragraph are presented from ISO (ISO 15686-1:2000).

| | |
|---|---|
| Design life | : (referred to Introduction in this paper) |
| Estimated service life | : Service life that a building or parts of a building would be expected to have a set of specific in-use conditions, calculated by adjusting the reference in-use conditions in terms of materials, design, environment, use and maintenance |
| Predicted service life | : Service life predicted from recorded performance over time |
| Forecast service life | : Service life based on either predicted service life or estimated service life |
| Factor method | : Modification of reference service life by factors to take account of the specific in use condition |

### 4.1 Role of service life estimation

Mutual relation among design life, service life estimation and service life prediction in view of forecasting the service life of whole building, a part of building etc. in the service life planning can be illustrated as Figure 3.

Among with increase of social interests and need, various prediction system or sub-systems have been investigated, developed, and also usable durability data have been collected, however rather limited ones have been applied to the service life planning for the limited building projects. It would be clear that one of the effective way to cope with the recent social requirement for service life issues of building should be improving the service life planning as to the design method even in the design for ordinary project. It would be also clear that further time would be needed to accomplish the service life planning system which depends fully upon the “prediction system,” here, the role of estimation system could be stressed.

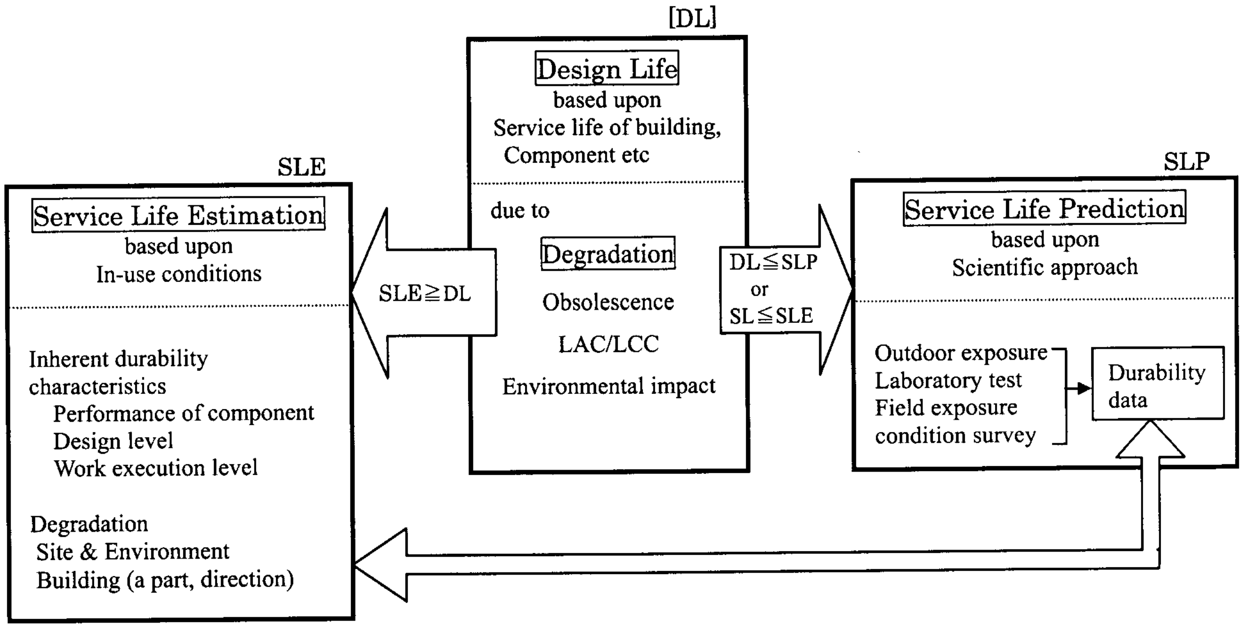


**Figure 3. Mutual relation among design life, service life estimation and prediction**

### 4.2 Factor method

The factor method specified in ISO (ISO 15686-1) was originally established in the Ministral R & D in Japan, which intended to support the estimation of design life by designer as the system enable to be applied throughout all objective from a building to material on the basis of huge amount of research and technical documents, condition survey and expert knowledge (agreed among designer, scientist, university, engineer, constructor, manufacturer).

This factor method could be established in reliance on the elementary systems like durability evaluation, estimation/prediction as shown in Figure 4, and the actual process flow for external coating system of reinforced concrete building is shown in Figure 5 as an example. In practice, the service life of external coating system can be estimated according to the simple equation as in Figure 5.

#### 4.2.1 Reference service life [Yo]

The reference service life for external coating systems (6 - 12 years for paint, masonry coating etc.) had been proposed in the Ministral R & D at first, and the original set has been revised with adding for the new products, like high performance or hybrid coating (15 years) since then.

Generally the priority of decision making for setting a reference service life would be the existing research data, Nireki et al. (1987) (1990), including condition survey at first and then, technical information from manufacturers, previous experience, and the final design would be remained long discussion to have an agreement among the experts from wider fields.

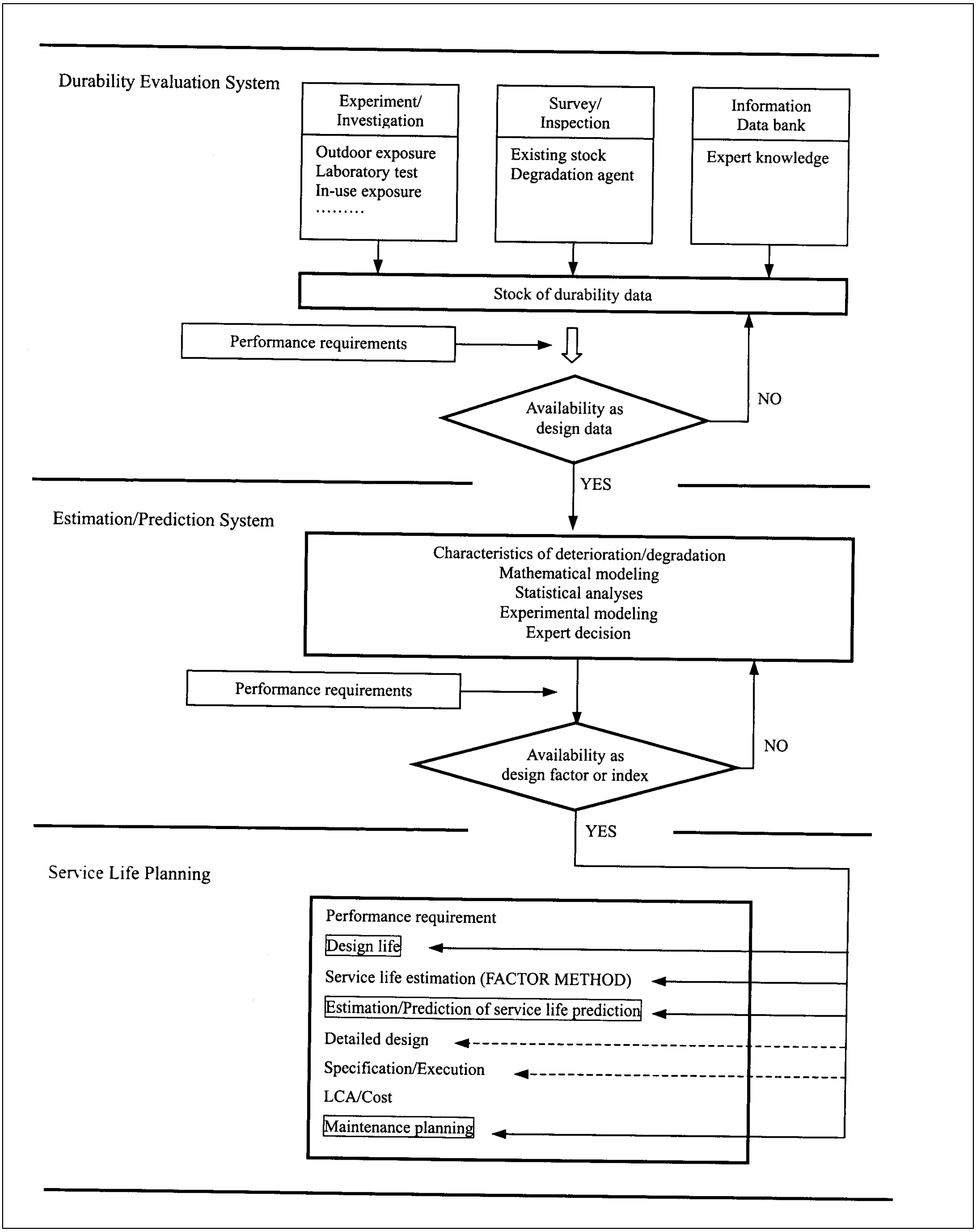


**Figure 4. Factor method and relevant elementary systems, modified Nireki (2000)**

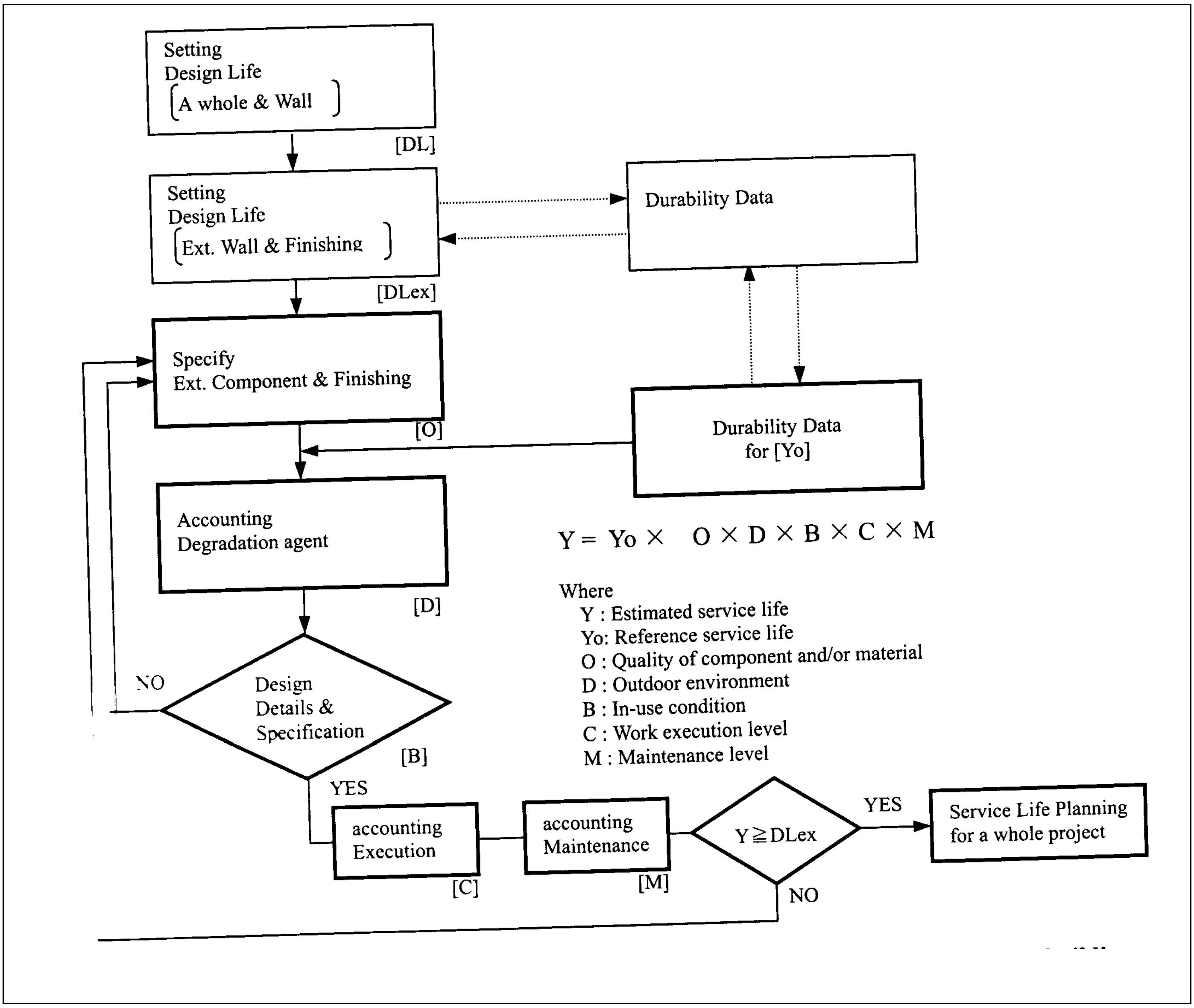


**Figure 5. Service life estimation flow for external coatings of reinforced concrete building modified Nireki (2000)**

4.2.2 Quality of component and/or material

For external coating, [B] can be obtained from the multiplied value of two sub-factors, the one; with or without top coating layer and the other; level of coating guide (for example the guides issued from the ministerial department, AIJ specification etc.: less than, according to, above the guide)

4.2.3 Outdoor environment [D]

Fairly much amount of durability data based on performance evaluation concept are available for external coating system Nireki et al. (1987)(1990), then, this factor can be obtained from the multiplied value of two sub-factors as shown Table 1, the one: weight of degradation agents [W], the other: degree of each degradation agents [X], then the outdoor environment [D] is according to as in Table 2. Generally this factor [D] is naturally varied from the objective as such steel, concrete, timber, waterproofing, hence, sometime it has more complicated sub-factors has to be taken into account. Of course indoor environment should be taken into account as a factor if the environment affects the external wall from the inside.

It would be notable that the knowledge or experience based information from practitioners could be useful to adjust research based sources.

**Table 1. Sub-factor for [D]**

| Items | Point W | Degree X 3 | Degree X 2 | Degree X 1 | Reference |
|---|---|---|---|---|---|
| Temperature (? ) | 1.5 | ~ 7.5 | 7.5 ~12.5<br>22.5 ~ | 12.5 ~ 22.5 | Yearly normal temperature |
| | | W·X=4.5 | 3.0 | 1.5 | |
| Humidity (%) | 1.5 | 80 ~ | 70 ~ 80 | ~ 70 | Yearly normal relative humidity |
| | | W·X=4.5 | 3.0 | 1.5 | |
| Rainfall (mm) | 2.0 | 3,500 ~ | 1,500 ~ 3,500 | ~ 1,500 | Yearly rainfall |
| | | W·X=6.0 | 4.0 | 2.0 | |
| Irradiation (kcal/m$^2$? day) | 2.0 | 3,300 ~ | 2,900 ~ 3,300 | ~ 2,900 | Total irradiation |
| | | W·X=6.0 | 4.0 | 2.0 | |

**Table 2. Outdoor environment factor [D]**

| Sub-factor (SW· X) | [D] |
|---|---|
| ~ 10 | 1.1 |
| 10 ~ 19 | 1.0 |
| 19 ~ 21 | 0.9 |

4.2.4 In-use condition [B]

The direction of building and the part of it would be the dominant sub-factors for external coating system, then, [B] can be obtained the orientation and the part where the coating system is applied as in Table 3. Much more complicate sub-factors are taken into account for timber construction, and also this factor [B] is exclusive in another objectives.

**Table 3. In-use condition [B]**

| Direction | Part of building: Horizontal salient Vertical salient | Part of building: Around opening Non-flat surface | Part of building: Ordinary external wall |
|---|---|---|---|
| West | 0.7 | 0.7 | 0.8 |
| North | 0.8 | 0.8 | 0.9 |
| South/East | 0.9 | 0.9 | 1.0 |

4.2.5 Work execution factor [C]

The workmanship and on-site management would be the dominant factors affecting the site work, hence the planning for execution and management, and on-site inspection system are selected. In conclusion, the level of work execution could be shown in Table 4.

**Table 4. Work execution factor [C]**

| | | Execution plan factor | | | | |
|---|---|---|---|---|---|---|
| | | 0.6 | 0.8 | 1.0 | 1.1 | 1.2 |
| Inspection factor | 0.6 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 |
| | 0.8 | 0.5 | 0.7 | 0.8 | 0.9 | 1.0 |
| | 1.0 | 0.6 | 0.8 | 1.0 | 1.1 | 1.2 |
| | 1.2 | 0.8 | 1.0 | 1.2 | 1.3 | 1.4 |
| | 1.4 | 1.0 | 1.3 | 1.4 | 1.5 | 1.6 |

The execution plan factor are in relevant to the fallowing several sub-factors as in Table 5 to 8. On the bases of Table 5 to 8, execution plan factor can be set as in Table 9.

**Table 5. Constructor/Manager Sub-factor**

| | Manager | | |
|---|---|---|---|
| | Not qualified | 2nd class architect*<br>2nd class manager* | 1st class architect*<br>1st class manager* |
| Not qualified | 0.7 | 0.9 | 0.9 |
| 2nd class painter* | 0.8 | 1.0 | 1.1 |
| 1st class painter* | 0.9 | 1.1 | 1.2 |

*Qualified by national qualification system

**Table 6. Terms of execution**

| Terms of execution | Point |
|---|---|
| Under inadequate ambient humidity | 0.7 |
| Under inadequate ambient temperature | 0.8 |
| Under appropriate condition | 1.0 |

**Table 7. Work process**

| Work process | Point |
|---|---|
| Inadequate | 0.7 |
| Slightly inadequate | 0.8 |
| Almost appropriate | 1.0 |
| Appropriate | 1.2 |

**Table 8. Work environment**

| Work environment | Point |
|---|---|
| Inadequate | 0.7 |
| Slightly inadequate | 0.8 |
| Almost appropriate | 1.0 |
| Appropriate | 1.2 |

**Table 9. Execution plan factor**

| Multiplied value in Table5.to8 | Point |
|---|---|
| 0.2 ~ 0.5 | 0.6 |
| 0.5 ~ 1.0 | 0.8 |
| 1.0 ~ 1.2 | 1.0 |
| 1.2 ~ 1.5 | 1.1 |
| 1.5 ~ | 1.2 |

On the other hand, inspection factor as in Table 4 related to the following several sub-factor as in Table 10. The inspection for external coating system would be quality of coating materials (conformity to specification, relevant quality standard, technical data from manufacture etc.) substrate (water content, alkaline, strength, cleanness etc.) and coating system (appearance, dryness, gloss, thickness etc).

The level and method of inspection can be summarized as in Table 10. In practice, all the item, totaled 14 items, are ranked as 2 to 3 degrees in each level I toIII, and the inspection factor can be obtained as in Table 11.

**Table 10. Outlook of inspection level and method**

| Item | | Inspection level | | |
|---|---|---|---|---|
| | | ? | ? | ? |
| Material [C] | Quality | Conformity | Test data | Authorized result |
| Substrate [S] | Water | Visual | Investigation | Measuring |
| | Roughness | Visual | Crack scale | Scaling |
| | Strength | Visual | Tape test | Tensile test |
| Coating [F] | Thickness | Use quantity | Thickness gauge | - |

**Table 11. Inspection factor**

| Item | | Inspection level | | | | |
|---|---|---|---|---|---|---|
| Material | Quality | - | I | I | II | III |
| Substrate | Water | I | II | III | III | III |
| | Alkali | I | II | III | III | III |
| | Absorption | - | I | I | II | II |
| Coating film | Roughness | I | I | II | II | II |
| | Soiling | I | I | I | I | I |
| | Strength | - | I | II | II | III |
| | Drying | - | - | I | I | I |
| | Appearance | I | I | II | II | II |
| | Gloss | - | I | II | II | III |
| | Thickness | I | I | I | I | II |
| | Hardness | - | I | I | I | I |
| | Adhesive | I | I | II | II | III |
| | Pin-hole | - | I | I | II | III |
| Inspection factor | | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 |

4.2.6 Maintenance factor [M]

The dominant element for maintenance would be the inspection work and period (cycle) of inspection in the maintenance planning. In conclusion, the level of maintenance [M] could be shown in Table 12.

**Table 12. Maintenance factor [M]**

| Value in Table 13. | Maintenance [M] |
|---|---|
| 0.18 | 0.7 |
| 0.3 ~ 0.42 | 0.8 |
| 0.5 ~ 0.72 | 0.9 |
| 0.8 ~ 1.0 | 1.0 |
| 1.2 ~ 1.44 | 1.1 |

**Table 13. Inspection cycle and factor**

| Inspection work factor | Inspection cycle factor | | | | |
|---|---|---|---|---|---|
| | 0.3 | 0.5 | 0.7 | 1.0 | 1.1 |
| 0.6 | 0.18 | 0.3 | 0.42 | 0.6 | 0.72 |
| 0.8 | 0.18 | 0.4 | 0.56 | 0.8 | 0.96 |
| 1.0 | 0.18 | 0.5 | 0.7 | 1.0 | 1.2 |
| 1.2 | 0.18 | 0.6 | 0.84 | 1.2 | 1.44 |

*0.3 in inspection cycle factor means without any inspection

The inspection cycle factor as in Table 13 can be obtained in Table 14.

**Table 14. Inspection cycle factor**

| Part of building | | Inspection cycle | | | | |
|---|---|---|---|---|---|---|
| Paint | Wall, Penthouse | - | 4 | 3 | 2 | 1 |
| | Others | - | 3 | 2 | 1 | 0.5 |
| Masonry coating | Wall, Penthouse | - | 4 | 3 | 2 | 1 |
| | Others | - | 3 | 2 | 1 | 0.5 |
| | Others | - | 3 | 2 | 1 | 0.5 |
| Inspection cycle factor | | 0.3 | 0.5 | 0.7 | 1.0 | 1.2 |

*0.3 means without any inspection

As to the inspection work factor, objective of inspection can be classified into as A (ordinary wall), B (balcony, eaves) and C (pitched wall, around openings, exposed column and beam, parapet etc) in the order of deterioration of coating systems, and the level of inspection also can be divided into as ? (by visual or contact (touch)), ? (measuring by using simple devices, like scale, cross cut test etc.) and ? (measuring by using specific device, like colour and gross meter, adhesive tester etc.). In Table 15, the inspection method for each degradation and level of inspection are shown as an example.

**Table 15. Inspection level and inspection method**

| Degradation | Inspection level | | |
|---|---|---|---|
| | I | II | III |
| Discolouration | Visual | Visual, Colour chart | Colour meter |
| Blister | Visual | Visual, for top coating | Visual, for top coating |
| | - | Visual, for core layer | Visual, for core layer |
| | - | Adhesive, Cross cut | Adhesive, Cross cut |

*Totaled 16 items have been provided

Then, inspection work factor can be obtained as in Table 16.

**Table 16. Inspection work factor**

| Inspection item | | Paint group | | | | Masonry coating | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | CD Discolouration | I | I | II | III | I | I | II | II |
| 2 | GD Decreasing gloss | I | I | II | II | I | I | II | II |
| 3 | CK Chalking | I | I | II | III | I | I | II | II |
| 4 | CT Soiling | I | I | II | II | I | I | II | II |
| 5 | BT Blister (Top coat) | - | I | II | II | - | - | - | - |
| 7 | CT Crack (Top coat) | - | I | II | II | - | - | - | - |
| 9 | ST Peel (Top coat) | - | I | II | III | - | - | - | - |
| 6 | BU Blister (Core layer) | - | - | - | | | I | II | III |
| 8 | CU Crack (Core layer) | - | - | - | - | - | I | II | III |
| 10 | SU Peel (Core layer) | - | - | - | - | - | I | II | III |
| 11 | M Mixture | - | - | - | - | - | I | II | III |
| 12 | W Decreasing thickness | - | I | II | III | - | I | II | III |
| 13 | CS Breaking (Core layer) | - | - | - | - | - | I | II | III |
| 14 | AS Breaking (Substrate) | - | I | II | II | - | I | II | II |
| 15 | CF Crack (Substrate) | - | I | II | II | - | I | II | II |
| 16 | FS Lifting (Mortar) | - | I | II | II | - | I | II | II |
| Inspection work factor | | 0.6 | 0.8 | 1.0 | 1.2 | 0.6 | 0.8 | 1.0 | 1.2 |

## 5 CONCLUSIONS

The factor method would remain its service life for a while, might be expected by the limit state when the service life prediction at present would be further improved and enable to cover the most objectives, and make the factor method as an obsolescent tool.

The factor method still has various aspects to be investigated toward farther practical application even though its parts have been adopted in the newly enforced Japanese code on housing systems, the concept of is in the main text and some details are in the evaluation standard in the enforcement document for the code, Motohashi & Nireki (2001)

*Limited or partial application* – A part or some parts of the factor method including the its concept can be of assistance in the various projects likely for setting a reference service life, ranking or rating of environment on site work, maintenance conditions, and such way of application could be expected to increase in different way of manners.

*Further investigation into the factor* – Periodical investigation into the already set and the new reference service life should be important to sustain the reliability of this method mainly based on the up to date durability data. Further examination of new factor or sub-factor like microclimate and GSI data into the environment factor, re-examination of work execution, much more consideration of the maintenance planning and sub-factors for maintenance level.

*New application to existing buildings* – At present, interest to the service life planning has been rather focused upon the new construction even ISO 15686-1 says that, it can be applicable both new and refurbishment of existing structures "with additional considerations may apply to existing building" in its scope. This additional considerations are not so simple as to just replacement of several terms and factors in the principles, in fact, there necessary to provide tactic measures and sequential technologies as such condition survey and diagnostics method to define the state of degradation, condition assessment to determine margine of service life, evaluation of repair material and/or replace component, service life estimation for refurbished parts and finally estimation or prediction of "residual service life".

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Above sequential technologies and techniques had already been investigated in the Ministral R & D, especially in the target [I] and [IV] as in the Ministral R & D, and various practical guides for existing stock had been established on the basis of the Ministral R & D. These guide were originally for government-owned buildings and nowadays these are referred to the public and privately-owned buildings Guide(1993), Guide(1999), Guide Note(1998). AIJ had also issued the relevant duide "Principal Guide for Condition survey, Diagnosis and Repair of Buildings" (Sub-committee, Durability), and the new guide "Principal Guide for Refurbishment of Buildings" will be issued within 2001.

One of the prime policy target of the MLIT in the fiseal year 2001 is activation of the used (second hand) housing market, this leads again to housing performance indication system for existing housing and also performance evaluation standard and criteria as almost similar track as of new housing, this relates to the forcul issue, residual service life of existing dwelling, often estimation would be required for the both before (at a certain time) and after the refurbishment (improved, up draded) of the dwelling in question.

To apply the factor method for existing building have already been experienced usually in conjunction with improvement or refurbishment project (for a building or a part of building), like repair and improvement work project for the external insulation waterproofing of publicly-owned multifamily flats; taking some practice as examples – as to decision making for the selection of new waterproofing to be replaced at first, execution work level, maintenance planning during the next time of refurbishment, as the new way of application of the factor method.

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# A Model For In-Ground Attack By Decay Fungi

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**Summary: This paper describes the development of a prediction model for in-ground attack of timber decay fungi. This model was initially based on in-ground tests on small clear wood specimens comprising some 80 species and 30 preservative treatments observed over 30 years around Australia. The effects of soil moisture content, important climate parameters such as rainfall and temperature, timber species, preservative treatment, and maintenance are considered. Collection of field data from real structures for model calibration is being under taken. Some preliminary calibration results and suggestions are presented.**

**Keywords: durability, timber, fungi, decay, climate, calibration.**

## 1 INTRODUCTION

Durability is one of the most important considerations in the use of timber in construction. Representatives of competing construction materials typically cite this as one of the ‘disadvantages’ of timber for certain applications compared to their products.

While many timber durability guidelines and design aids are available to engineers and designers, the current approach to durability design is very much an art; design solutions vary from person to person and control of performance depends almost exclusively on compliance to good building practice. This tends to inhibit innovation and optimisation of building design, and is not useful for assessing the impact of rapid changes enforced by legislation (e.g., banning of chemicals). To address these issues, a major multi-disciplinary project in Australia primarily funded by the Forestry and Wood Products Research and Development Corporation (FWPRDC) and various research organisations (CSIRO, State Forests of New South Wales and Queensland Department of Primary Industries, Forest Research Institute) was initiated several years ago to develop an engineering approach to design durability into timber construction (MacKenzie 1996; Foliente *et al.* 1999). This effort requires understanding of the degradation mechanisms in timber construction, establishing a durability database, developing durability prediction models, and calibrating these models to the available knowledge base, including laboratory and field data.

The strength models have been based on a study of the decay of small clear pieces of wood (Thornton *et al.* 1997; Leicester *et al.* 2001) exposed to in-ground soil contact for a period of about 31 years. Because this exposure is for a relatively short time and does not cover all aspects of the exposure of real construction, it will be necessary to calibrate and further develop these models using data obtained from full size timber construction, before they can be applied to practical design. This paper describes the development and calibration of models for predicting the decay and degradation of timber in ground contact.

## 2 PREDICTION MODEL FOR DECAY

### 2.1 Soil moisture content

Moisture is a primary factor in promoting the decay of wood. It is recognised that if timber is in contact with a soil that has a soil suction value of less than 1.5 MPa, then it will eventually come to equilibrium at a moisture content above the fibre saturation point and the timber will then decay quite quickly.

To investigate the effect of soil moisture, some simple computations were undertaken. One set of results is shown in Table 1. These computations were made using simple Green-Ampt procedures (Jury *et al*. 1991) applied to ideal soils with properties taken from Rawls *et al*. (1991). They show the time taken for an exposed expanse of soil to dry after a brief rainfall lasting less than an hour. A soil is deemed to be “dry” when the soil moisture suction is larger than 1.5 MPa. Rainfall ponding effects are taken into account. It is assumed that the water table is effectively infinitely deep and that the soil is dry when the rain

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commences. It is to be noted that for most cases, the ground will remain continuously wet unless the rainfall is less than something like 5 mm per month.

**Table 1. Time taken to dry the soil after a single rainfall**

| *SOIL* | *Time to dry soil (days)* | |
|---|---|---|
| | *Rain = 25 mm* | *Rain = 5 mm* |
| 1. SAND | * | 153.4 |
| 2. loamy SAND | * | * |
| 3. sandy LOAM | * | 38.5 |
| 4. LOAM | 72.5 | 9.5 |
| 5. silt LOAM | 9.5 | 1.5 |
| 6. silty clay LOAM | 81.5 | 3.5 |
| 7. sandy clay LOAM | 316 | 15.5 |
| 8. clay LOAM | 33.5 | 1.5 |
| 9. sandy CLAY | * | 65.5 |
| 10. silty CLAY | 59.5 | 3.5 |
| 11. CLAY | 74.5 | 9.5 |

## 2.2 Climate index

It is known that the most important climate parameters affecting in-ground timber decay are the moisture content and temperature of the wood. Consequently the parameters chosen to provide a climate index for decay are

- mean annual rainfall
- number of months with rainfall less than 5 mm
- mean annual temperature

Figure 1 shows the number of months in which rainfall < 5 mm at Bureau of Meteorology data sites in Australia.

By calibration with measured depths of in-ground decay, the following procedure was developed for computing a climate index for this decay.

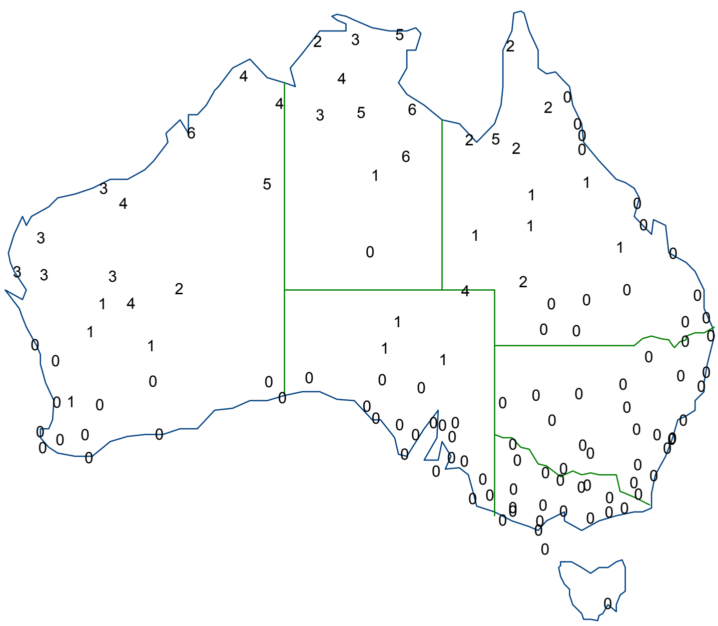

**Figure 1. Map showing the number of months in which mean annual rainfall is less than 5 mm.**

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First a function considering the effects of rainfall and number of dry months, $f(R)$ is proposed as follows,

$$f(R)=\begin{cases}0 & \text{if } R \le 250 \text{ mm or} \\ 10\left[1-e^{-0.001(R-250)}\right] & \text{if } R > 250 \text{ mm}, N_{\mathrm{dm}}=0 \\ 10\left[1-e^{-0.001(R-250)}\right]\left(1-\dfrac{N_{dm}}{6}\right) & \text{if } R > 250 \text{ mm}, N_{\mathrm{dm}}>0.\end{cases} \tag{1}$$

where $R$ is the mean annual rainfall, and $N_{dm}$ denotes the number of dry months per year.

Next a function considering the effect of temperature, $g(T)$ is defined by

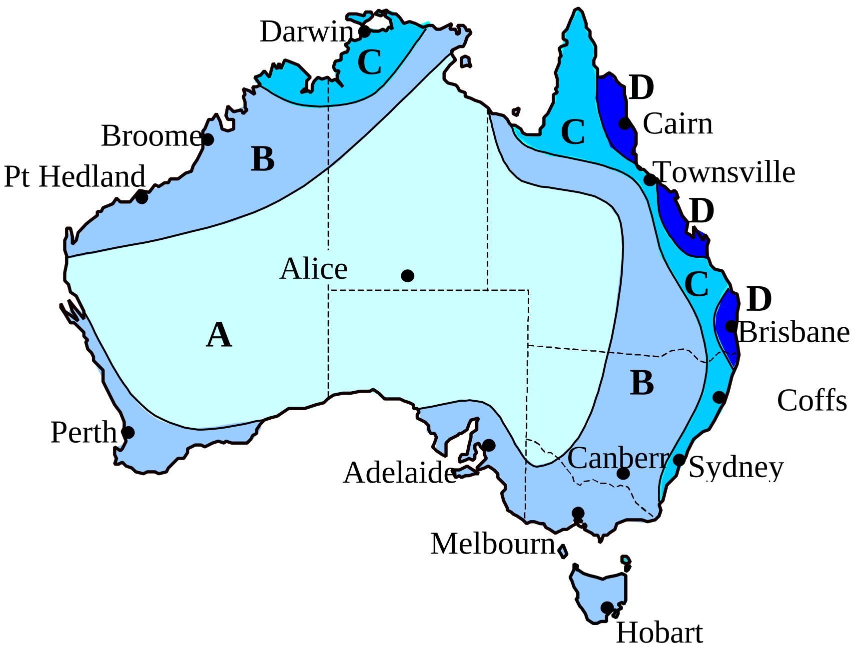
$$g(T)=\begin{cases}0 & \text{if } T \le 5^{\circ}\text{ C}, \\ -1+0.2T & \text{if } 5 < \mathrm{T} \le 20^{\circ}\text{ C}, \\ -25+1.4\mathrm{T} & \text{if } T > 20^{\circ}\text{ C}.\end{cases} \tag{2}$$

where $T$ is the mean annual temperature.

Functions $f(R)$ and $g(T)$ are then used to determine a climate index $I_{ig}$ as follows

$$I_{ig} = f(R)^{0.3}\, g(T)^{0.2} \tag{3}$$

An in-ground hazard map for Australia, with climate zones related to the computed climate index $I_{ig}$, is shown in Fig. 2.




| Hazard Zone | Climate Index $I_{ig}$ |
|---|---|
| A | 0.5 |
| B | 1.5 |
| C | 2.0 |
| D | 3.0 |

Zone D has the greatest decay potential

**Figure 2. In-ground decay hazard zones for Australia.**

### 2.3 The computational model

On the basis of the data from in-ground tests on small samples of outer heartwood, an idealised quantitative model for the development of decay was chosen. This model is illustrated in Fig. 3 and is assumed to apply to all other cases, such as for corewood and sapwood as shown in Fig. 4. The model indicates that the progress of decay is characterised by a lag time and a uniform rate of decay. For many cases, it is convenient to replace the rate of decay by a parameter $d_{20}$, the depth of decay after 20 years. The effect of maintenance is assumed to be the introduction of a lag as illustrated in Fig. 3.

In order to estimate the loss of element strength due to decay it is assumed that decayed wood has no strength and un-decayed wood maintains its original strength. Hence it is necessary not only to make an estimate of the rate of decay, but also to select appropriate attack patterns for the decay. Combination of these two factors then enables a calculation to be made of the residual cross-section and hence the residual strength of an element after a specified in-service duration. Some suitable components of decay attack patterns are shown in Fig. 5.

In the following sections, the subscripts used have the following meanings:

- 'core' = corewood
- 'heart' = outer heartwood
- 'lag' = lag time
- 'sap' = sapwood
- 'treat' = treated timber
- 'un' = untreated timber
- '20' = the decay after 20 years
- 'dc4' = value for timber of durability class 4.

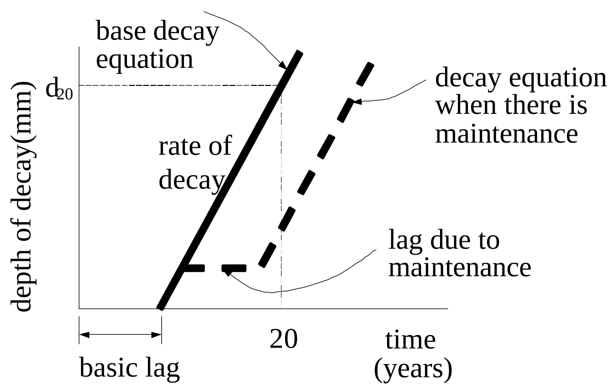



**Figure 3. Idealised model for attack by decay.**

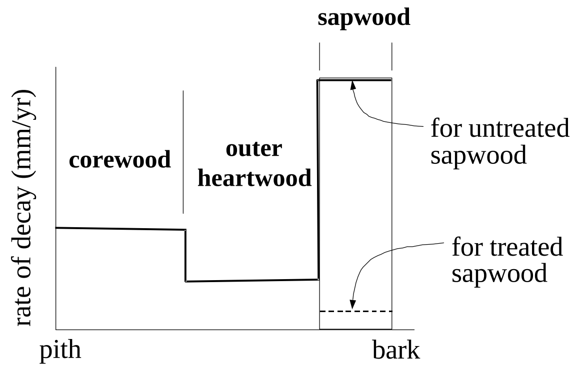



**Figure 4. Schematic illustration for the rate of decay within a pole.**

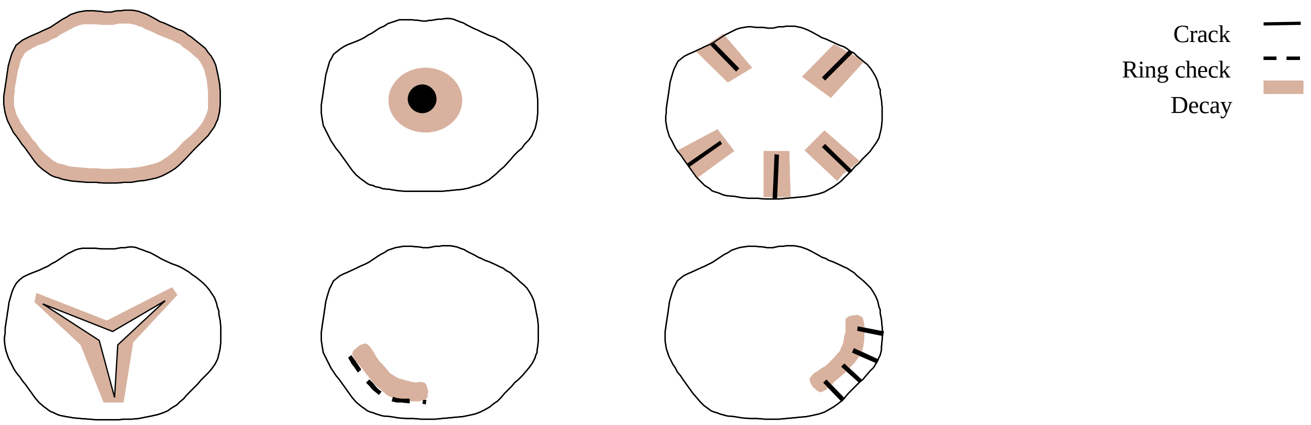



**Figure 5. Examples of components of decay attack patterns.**

## 2.4 Untreated timber

### 2.4.1 Outer heartwood

The decay at 20 years, $d_{20,un,heart}$, and the rate of decay, $r_{un,heart}$, are given by

$$d_{20,un,heart} = AI_{ig} \tag{4}$$

$$r_{un,heart} = \frac{AI_{ig}}{20 - t_{lag,un,heart}} \tag{5}$$

where $A$ is a constant and $t_{lag,un,heart}$ is the time lag to decay for untreated heartwood timber.

The values of $A$ and $t_{lag,un,heart}$ are tabulated in Tables 2 and 3 respectively. To use these tables it is first necessary to classify a timber into a durability class; this can be done according to the classifications of Thornton *et al* (1997).

**Table 2. Decay parameter A for untreated heartwood timber**

| *Natural Durability class* | *A* |
|---|---|
| Class 1 | 3 |
| Class 2 | 7 |
| Class 3 | 11 |
| Class 4 | 30 |

**Table 3. Basic lag time for outer heartwood**

| *Treatment of external sapwood* | *Durability class* | *Lag for heartwood timber attacked from centre (years)* | *Lag for heartwood timber attacked from outside after sapwood has decayed (years)* | *Lag for heartwood timber attacked from outside for desapped pole (years)* |
|---|---|---|---|---|
| *None* | 1 | 15 | 0 | 6 |
| *None* | 2 | 8 | 0 | 4 |
| *None* | 3 | 5 | 0 | 2 |
| *None* | 4 | 2 | 0 | 1 |
| Treated | 1 | 25 | 0 | NA |
| Treated | 2 | 20 | 0 | NA |
| Treated | 3 | 15 | 0 | NA |
| Treated | 4 | 10 | 0 | NA |

### 2.4.2 Corewood

For this wood the lag time to decay and the decay rate after the lag time has elapsed are given by

$$t_{lag,un,core} = t_{lag,un,heart} / m \tag{6}$$

$$r_{un,core} = m r_{un,heart} \tag{7}$$

where $m$ is a relative decay rate factor. A value of $m = 2$ is recommended as an initial choice for calibrating the model with field data.

If required, the decay at 20 years may be determined by

$$d_{20,un,core} = \left(20 - t_{lag,un,core}\right) r_{un,core} \tag{8}$$

### 2.4.3 Sapwood

The time lag is taken as

$$t_{lag,un,sap} = 0.5 t_{lag,un,dc4} \tag{9}$$

The decay at 20 years is determined by

$$d_{20,sap} = 1.5 d_{20,un,dc4} \tag{10}$$

### 2.5 Treated timber

To compute $d_{20,tr}$, the 20-year decay of treated timber, the value of $d_{20,un}$ (the 20-year decay of the timber if untreated) and $d_{20,un,dc4}$ (the 20-year decay of Class 4 untreated timber) must first be computed using Eq. (4). The value of $d_{20,tr}$ is then determined by

$$d_{20,tr} = \min\left[ d_{20,un}, \quad \frac{d_{20,un,dc4}}{1 + BR_{CCA-equiv}} \right] \tag{11}$$

where $B$ is given by

$$B = \begin{cases} 7400/d_{20,un,dc4}, & \text{for softwoods;} \\ 1500/d_{20,un,dc4}, & \text{for hardwoods.} \end{cases} \tag{12}$$

and R is the retention of a preservative treatment is expressed in terms of its CCA-equivalent retention, $C_{CCA-equiv}$, in kg/kg (%).

For the case of creosote, the CCA-equivalent retention is determined by

$$C_{CCA-equiv} = 0.018 C_{creosote} \tag{13}$$

where $C_{creosote}$ is the creosote retention in kg/kg (%).

If retention is given in kg/m$^3$, the conversion between kg/kg (%) and kg/m$^3$ may be done by

$$\text{kg/kg(\%)} = \begin{cases} \dfrac{0.36 \times \text{kg/m}^3}{D} \times 100, & \text{for CCA treatment;} \\ \dfrac{\text{kg/m}^3}{D} \times 100, & \text{for creosote treatment.} \end{cases} \tag{14}$$

in which $D$ is the air-dry density of timber (kg/m$^3$).

The lag time for treated timber is determined by

$$lag_{tr} = \max\{ lag_{un}, 8C_{CCA-equiv} \} \tag{15}$$

### 2.6 Maintenance

The extra lag time in the decay process that is due to the application of maintenance treatments is given in Table 4.

**Table 4. Lag time due to each application of a maintenance procedure**

| *Procedure on poles* | *Extra lag for heartwood timber attacked from centre (years)* | *Extra lag for heartwood timber attacked from outside after sapwood has decayed (years)* | | *Extra lag for heartwood timber of de-sapped poles attacked at time zero from outside (years)* | *Extra lag for sapwood timber attacked from outside (years)* |
|---|---|---|---|---|---|
| | | *Procedure at time zero* | *In-service procedure* | | |
| External diffusing chem.Barrier | 0 | 0 | 3 | 5 | 5 |
| External non-diffusing chem. Barrier | 0 | 0 | 0 | 3 | 2 |
| Insertion of diffusing material | 5 | 5 | 0 | 0 | 0 |

# 3 MODEL CALIBRATION

## 3.1 General

Mathematical models of durability need to be calibrated by in-service performance data. These take into account in the most realistic way both the failure patterns and long duration effects. In addition, it is useful to take into account the opinions of people who have practical experience and/or expertise in the durability performance of timber.

## 3.2 Data base

A challenge is the establishment of databases that capture all the required information needed for model development and calibration. To date, timber specimens have been collected from the states of Queensland, New South Wales and Victoria. These comprise power poles, fence posts and house stumps. As far as possible, timber is collected in the form of an assembly of elements, such as the fence assembly shown in Fig. 6. Once delivered at the laboratory, the timber is examined for decay and mechanical degradation. The measured data is then entered into a durability database, which was developed using Microsoft Access. Data on approximately 400 in-ground sections are available at present in the database. The in-service age of the samples range from 15 to 75 years.

## 3.3 Calibration with data from the Wedding Bells test site

Of particular interest is a sample of more than 60 poles from the Wedding Bells test site near Coffs Harbour, New South Wales(Gardner *et al* 1998),. These poles were monitored over periods of 18 to 25 years by researchers from State Forests of New South Wales. They comprised untreated poles of durable species, and poles treated with either CCA or creosote. In addition, many poles were subjected to in-situ maintenance treatments during their service life. Because monitoring of the performance of these poles was carefully done, the data obtained is particularly useful for calibration purposes.

Figure 7(a) shows the measured and predicted perimeter decay of untreated de-sapped durable poles. A wide scatter of the test data is seen to occur. This scatter must be taken into account in a reliability analysis to obtain design strengths for these poles. To see the trend of the measured versus predicted decay, the data are then grouped for timber of same durability class, same service life, and same type of external treatment, as shown in Fig. 7(b). It is seen that on average the assumed parameters appear to give predictions that match the field data reasonably well.

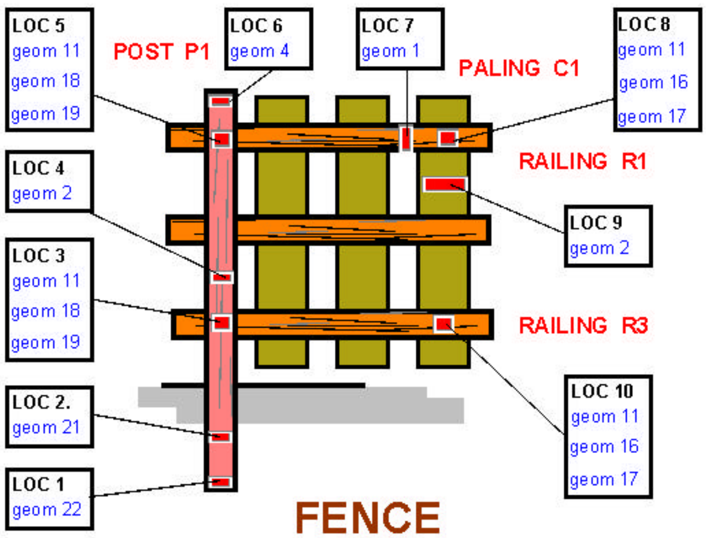
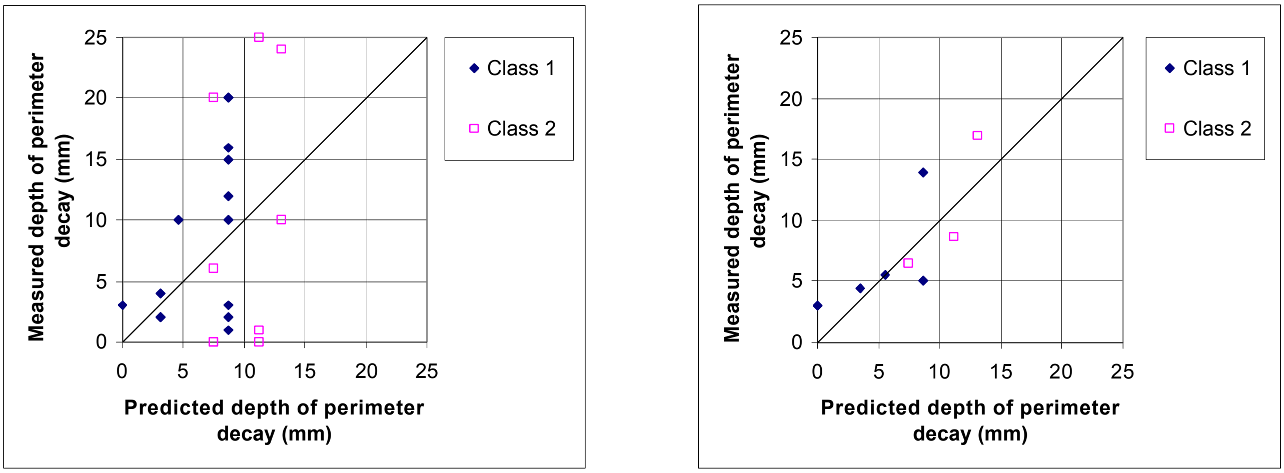


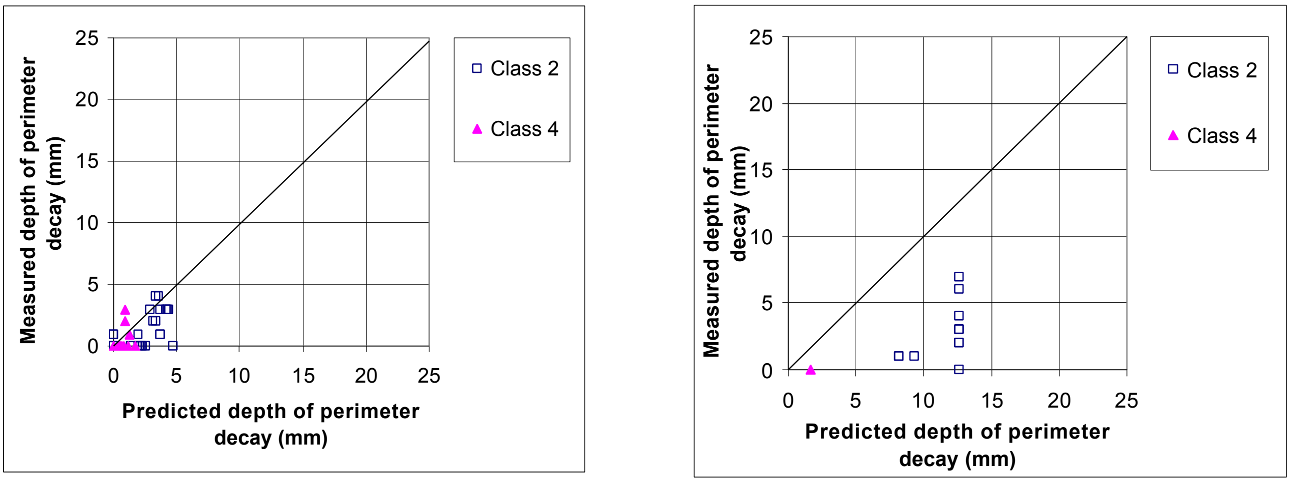

**Figure 6. Example of a fence assembly showing locations at which the progress of decay are measured.** ***(In-ground sections are taken from Locations Loc. 1 and Loc. 2).***




(a) scatter data (b) grouped data

**Figure 7. Perimeter decay of untreated de-sapped durable timber poles.**

As another example, Fig. 8 shows the measured and predicted perimeter decay of CCA and creosote treated poles. It is seen that the CCA data is roughly in line with the predictions whereas the decay of creosote treated timber appears to be over-estimated by a factor of about 3. This mismatch may be related to the fact that the retention of creosote is in fact a guess and was not recorded at the time of installation.




(a) CCA treated poles (b) creosote treated

**Figure 8. Perimeter decay of treated poles with maintenance treatment.**

### 3.4 Full size pole strength data

Hourigan *et al*. (1999) have strength tested full size poles after several years of service. These poles were CCA treated Spotted Gum poles used for electrical energy distribution. The results of these tests are shown in Fig. 9. For comparison, Fig. 10 gives examples of computed strength-time predictions of CCA treated spotted gum poles. The poles are assumed to be circular with a diameter of 400 mm, having a 20 mm sapwood treated with a 1% kg/kg loading of CCA preservative. For the durability graph shown in Fig. 10(a) the attack pattern was assumed to comprise centre rot and perimeter decay; and for Fig. 10(b) the pattern was assumed to be internal half-width cross cracks and perimeter decay. The shape of the strength-time relationships shown in Figs 9 and 10 differ and further investigations are required to resolve the reasons for this difference. However, the field data comes from four different populations of timber, and these may not have all been in service at the same locations. If so, this factor may have distorted the apparent trend of the measured data.

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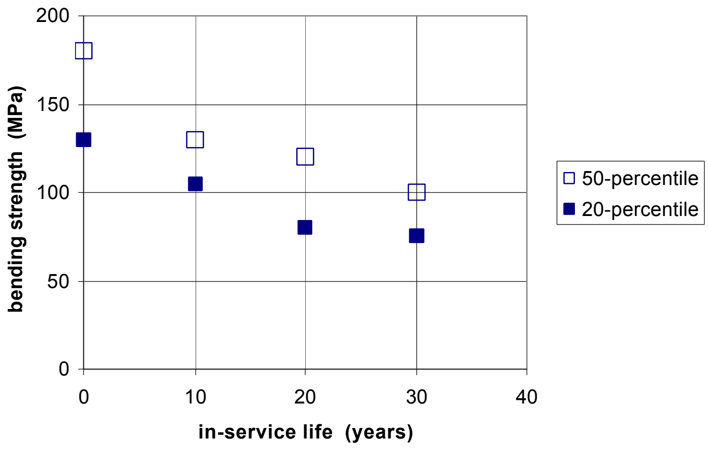
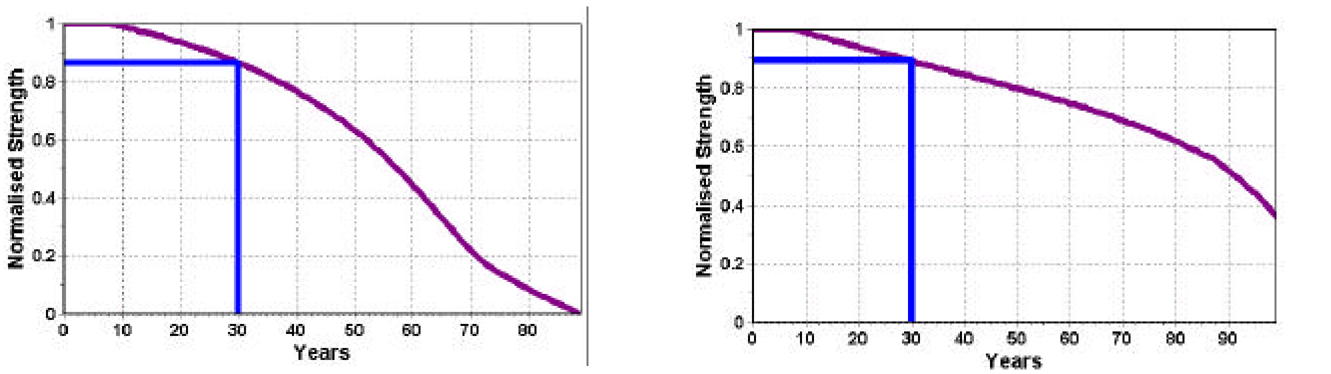



**Figure 9. Effect of time on the strength of CCA-treated Spotted Gum poles (Hourigan *et al*. 1999).**




(a) for assumed centre rot and perimeter decay patterns (b) for assumed half-width crack and perimeter decay patterns

**Figure 10. Predictions of pole strength over time (50-percentile estimate, assuming no maintenance procedures applied).**

## 4 CONCLUSIONS

Durability prediction models for timber in ground contact have been developed using data from in-ground testing of small clear pieces of wood. Calibration of these models is being undertaken using data obtained from field measurements of real in-service construction. The field data has been entered into a computer database. To date data on some 400 in-ground sections of wood from real structures have been collected. These samples have come from house stumps, fences, poles and posts obtained from Vistoria, New South Wales and Queensland.

## 5 ACKNOWLEDGEMENTS

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# A Model For Termite Hazard In Australia

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**Summary: A probability model to predict the time for termite attack on timber construction is developed. It makes use of the result from two previous studies, one from a collection of expert opinions, which provides a basis for establishment of statistics of four sequential event times, and the other from a termite tally for houses around Australia The tally gives data needed for identifying important parameters and constructing a hazard map as well as calibrating the proposed probability model. It represents the world's first predictive model of such an attack. Computer software has been written based on the model. It can be used for collecting additional expert opinions and for making risk predictions on termite attack. Application of the model indicates that the range of possible risks is so great that it is unrealistic to treat all houses as the same when assessing the impact of termites.**

**Keywords: termite, risk, hazard, probability, timber.**

## 1 INTRODUCTION

Durability is one of the most important considerations in the use of timber in construction. Consumers and asset owners are concerned about effective service life and overall maintenance costs. To address the concerns of the wider community, inclusion of durability requirements in the future Building Code of Australia is currently being seriously considered by building code officials.

This paper considers probabilistic modelling of termite attack, one of the most important durability issues for timber construction. The model developed is based on two previous studies. The first is a survey of some 5000 houses around Australia and provides statistical data on house location, age, termite incidence inside the house and in the garden, and construction type (Cookson 1999). This survey will be referred to as the "Termite Tally". The second study is a collection of expert opinions on termite behaviour, referred to as the "Expert Opinion" model; it provides a quantitative estimate by experts of the mean and variability of observed times of termite attack (Leicester and Wang 2001). The two studies are brought together in the form of a probabilistic model of termite attack on housing.

## 2 PROBABILITY MODEL

In the following, a probability model is used to estimate a "true" risk and an "apparent" risk. The apparent risk is the risk estimate based on the historical memory of the house occupant. In the Termite Tally the average time of occupancy of the householders interviewed was found to be about 11 years. In this study the historical memory of the average occupant, denoted by $t_{mem}$, has been taken to be 20 years.

The probability density function of the time for a house to be attacked by termites is assumed to be of the type shown in Fig. 1. The equation for the density function is assumed to be

$$p = a + bt \qquad (1)$$

where $a$ and $b$ are the distribution parameters, and $t$ is the time since time zero, the time at which the house was constructed. The value of $a$ may be either positive or negative, as shown in Fig. 1. The notation $t_{max}$ is used to denote the upper limit of the density function, evaluated from the assumption that the area under the density graph must be unity.

For the case of $a \geq 0$, the true probability that the house has been attacked by time $t$, denoted by $P(house,attack,true,t)$, is given by

$$P(house, attack, true, t) = \begin{cases} at + \frac{1}{2}bt, & \text{for } t \leq t_{max}; \\ 1, & \text{for } t > t_{max}. \end{cases} \qquad (2)$$

and the average time to the attack, denoted by mean($t$), is given by

$$\text{mean}(t) = \frac{a}{2} t_{\max}^2 + \frac{b}{3} t_{\max}^3 \qquad (3)$$

For the case of $a < 0$, the true probability that the house has been attacked by time $t$ is

$$P(house, attack, true, t) = \begin{cases} 0, & \text{for } t \le t_a; \\ a(t - t_a) + \frac{1}{3}\left(t^2 - t_a^2\right), & \text{for } t_a < t < t_{\max}; \\ 1, & \text{for } t \ge t_{\max}. \end{cases} \qquad (4)$$

and the average time of attack is

$$\text{mean}(t) = t_{\max} - \tfrac{1}{3}\left(t_{\max} - t_a\right) \quad (5)$$

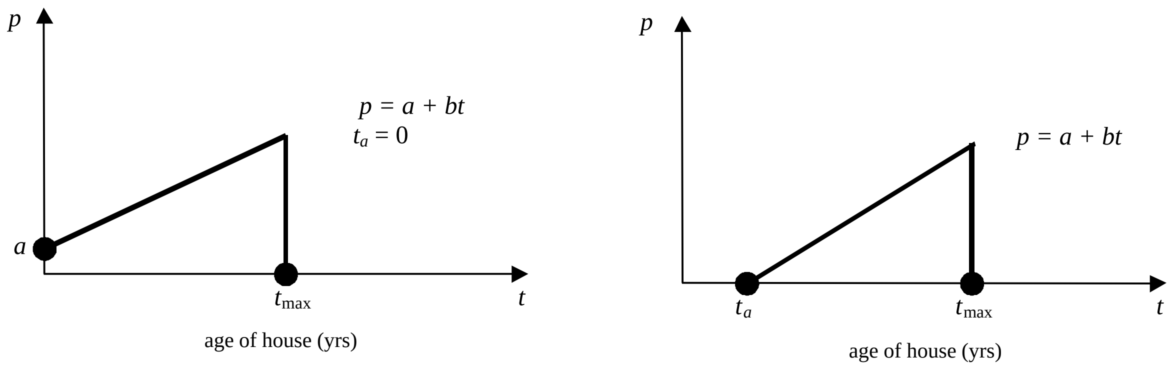
The apparent risk of attack, denoted by $P(house,attack,obsv,t)$, is the same as the true risk for the case $t < t_a + t_{mem}$, where $t_a$ denotes the start of the probability density functions shown in Fig. 1. For larger values of $t$ the apparent risk is given by

$$P(house, attack, obsv, t) = A + Bt \qquad (6)$$

in which

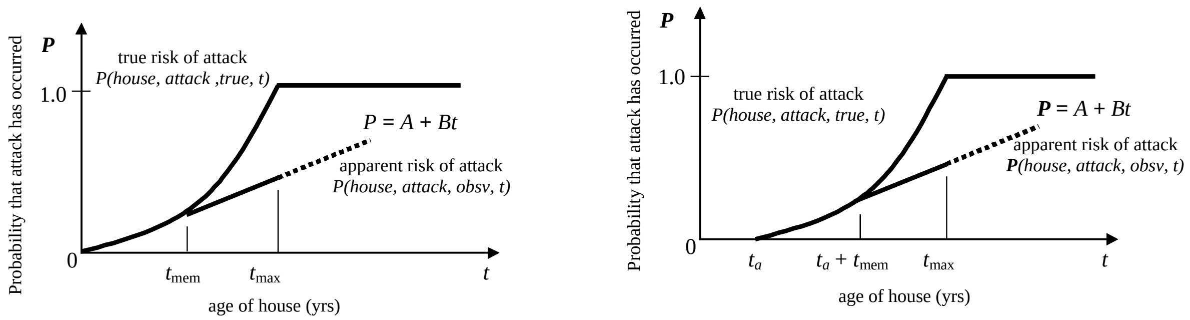
$$\begin{aligned} A &= at_{\text{mem}} - \tfrac{1}{2} bt_{\text{mem}}^2 \\ B &= bt_{\text{mem}} \end{aligned} \qquad (7)$$

Graphs for the apparent and true risks of attack are illustrated schematically in Fig. 2. In the following sections the proposed model will be verified and calibrated by the Expert Opinion and Termite Tally.




(a) for positive *a* (b) for negative *a*

**Figure 1. Probability density functions of the time of a termite attack.**




(a) for positive *a* (b) for negative *a*

b

**Figure 2. Schematic illustration of the cumulative distribution functions of the attack time.**

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## 3 THE EXPERT OPINION MODEL

### 3.1 The base model

The time estimate model based on expert opinions applies to the configuration illustrated in Fig. 3. It relates to a taget house surrounded by 50 m of termite-free land. The distance of 50 m was chosen because this is about the limit of the foraging distance of most termite species. The model used then endeavours to estimate four sequential event times (see Fig. 4) defined as follows:

1. time t1: the time taken for the establishment of a mature colony within a distance of 50 m from the target house;
2. time t2: the time taken for the termite foraging galleries to progress to a house 20 m away from the nest site;
3. time t3: the time taken for termites to penetrate or bypass a chemical or mechanical barrier, if any;
4. time t4: the time taken (after penetrating the barrier) to reach and cause failure of a timber member.

Relevant data on these four event times were obtained via a limited survey of expert opinion. Details of this survey, together with the analysis procedure used to process the data has been described elsewhere (Leicester and Wang 2001, Leicester *et al.* 2001).

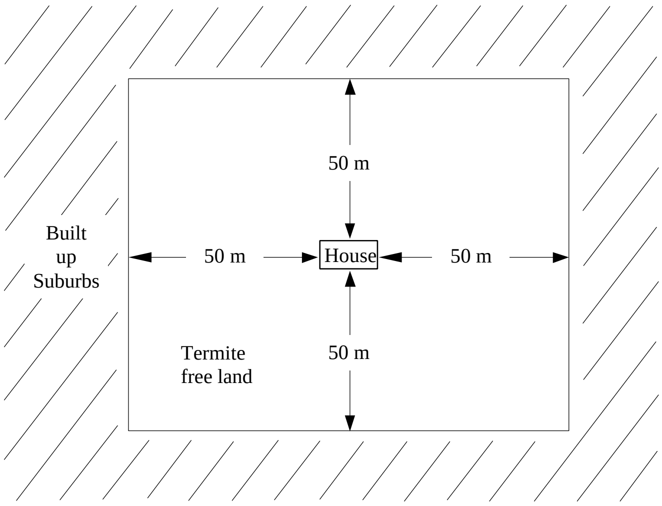


**Figure 3. Hypothetical scenario for house and land at time zero.**

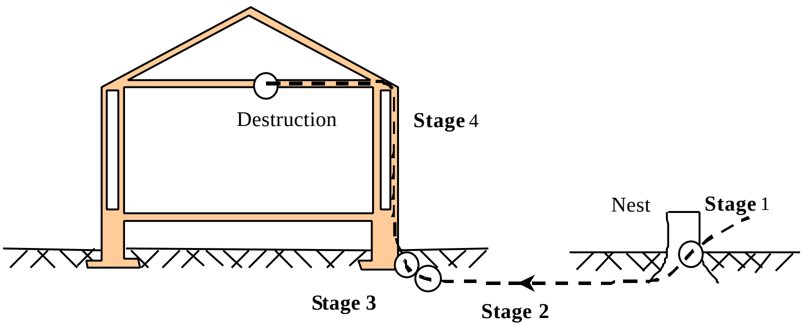


**Figure 4. Illustration of termite progress.**

In the survey, a set of parameters affecting each event time was obtained from experts. The set chosen is listed as $P_1$, $P_2$, …, $P_{14}$, as tabulated in Table 1. For each parameter, the experts were asked to list the importance of the parameter with regard to its influence on the relevant event time; this importance was rated on a scale of 1–10, with 10 being the most important; examples of the importance parameters chosen are also given in Table 1.

**Table 1. List of event times and associated parameters**

| *Event time* | *Influencing parameter* | *Importance factor* |
|---|---|---|
| $t_1$ Time for establishment of a colony | $P_1$: geographical location | 8 |
| | $P_2$: age of surrounding suburbs | 5 |
| | $P_3$: number of potential nest sites | 9 |
| $t_2$ Time for termites to travel to the building | $P_4$: geographical location | 8 |
| | $P_5$: soil condition | 6 |
| | $P_6$: food source | 7 |
| $t_3$ Time for penetration or bypass of a termite barrier* | $P_7$: geographical location | 4 |
| | $P_8$: period between inspections | 10 |
| | $P_9$: maintenance parameter | 7 |
| $t_4$ Time to reach and destroy timber | $P_{10}$: geographical location | 8 |
| | $P_{11}$: ground-contact building element | 5 |
| | $P_{12}$: period between inspections | 9 |
| | $P_{13}$: type of material attacked | 7 |
| | $P_{14}$: timber environment | 7 |

* Barriers considered were toxicant and repellant chemical barriers, Granitgard and Termimesh

For each parameter $P_j$, there is an associated parameter factor $k_j$. This factor $k_j$ is given the value of +1, 0 or –1 depending on whether the parameter has been chosen to correspond to low, medium or high termite activity respectively. For example, Tables 2 and 3 show how the parameter factor $k_3$, associated with parameter $P_3$, is chosen. This parameter relates to the time for establishment of a termite's nest by assessing the number of potential nesting sites within a 50 m distance of the target house.

**Table 2. Examples of potential nesting sites**

The following refers to potential nest sites for colonies of termites

- Tree *(diameter larger than* 300 mm*)*
- Tree stump or untreated pole *(diameter larger than* 200 mm*)*
- Untreated landscape timber *(e.g. sleepers, retaining walls of length* >1.0 m, *height* >0.5 m*)*
- Woodheap*(height* >0.5 m, *ground contact area* >0.5 × 0.5 m, *length of periods that bottom layer of woodheap is untouched* >1 *year)*
- Compost heap
- Wood 'stepping stones'
- Subfloor storage *(height* >0.5 m, *ground contact area* >0.5 × 0.5 m, *length of period which it is untouched* >1 *year).*
- Solid infill under a verandah
- Any part of a building with water continuously leaking into it.

**Table 3. Parameter $P_3$ indicating the effect of the number of potential nesting sites on time to establish a mature colony of termites**

| *Number of potential nesting sites* | *Effect on speed of establishment of a termite colony* | *Parameter factor* |
|---|---|---|
| <2 | Low | $k_3 = +1$ |
| 2–5 | Medium | $k_3 = 0$ |
| <5 | High | $k_3 = -1$ |

From the analysis procedure, the statisitics of an event time $t_i$ can be evaluated from

$$t_i = M_i A_i \qquad (8)$$

where $M_i$ is a constant and $A_i$ is a random variable.

In particular the four mean values, denoted by mean($t_i$), are given by

$$\text{mean}(t_i) = M_i \text{ mean}(A_i) \quad (9)$$

The mean values and coefficients of variation of the random variables $A_i$ are given in Table 4. The parameters $M_i$ are given by

$M_1$ = (1 + 0.236 $k_1$) (1 + 0.148 $k_2$) (1 + 0.266 $k_3$)

$M_2$ = (1 + 0.0771 $k_4$) (1 + 0.0578 $k_5$) (1 + 0.0675 $k_6$)

$M_{3A}$ = (1 + 0. 454$k_7$) (1 + 0.649 $k_8$)

$M_{3B}$ = (1 + 0. 497$k_7$) (1 + 0.710 $k_8$) (10)

$M_{3C}$ = (1 + 0.222 $k_7$) (1 + 0.556 $k_8$) (1 + 0.389 $k_9$)

$M_{3D}$ = (1 + 0.205 $k_7$) (1 + 0.513 $k_8$) (1 + 0.359 $k_9$)

M4 = (1 + 0.455 $k_{10}$) (1 + 0.284 $k_{11}$) (1 + 0.512 $k_{12}$) (1 + 0.398 $k_{13}$) (1 + 0.398 $k_{14}$)

Note that there are five values of $M_3$ and $A_3$ depending on the type of termite barrier used. The subscripts used refer to the type of barrier as follows:

*A*: granite-guard,

*B*: termimesh,

*C*: toxicant chemical,

*D*: repellant chemical

*E*: no barrier.

**Table 4. Distribution parameters of $A_i$**

| *Time* | *Mean($A_i$) (years)* | *COV($A_i$)* |
|---|---|---|
| $t_1$ | 16.7 | 0.646 |
| $t_2$ | 4.9 | 0.813 |
| $t_{3A}$ | 20.9 | 0.775 |
| $t_{3B}$ | 27.4 | 0.768 |
| $t_{3C}$ | 37.9 | 0.675 |
| $t_{3D}$ | 24.2 | 0.750 |
| $t_{3E}$ | 0 | 0 |
| $t_4$ | 14.5 | 0.625 |

### 3.2 The practical model

For practical application, the base model must be modified so that the target house is closer to the suburbs than 50 m. In addition, the model must allow for the possibility that there may be mature nests at time zero, the year in which the house is constructed. These modifications are illustrated in Fig. 5. A timeline of events is illustrated in Fig. 6. A rough approximation to the estimate of the attack time by an expert, denoted by $t_{\text{expert}}$, is

$$t_{\text{expert}} = P_{garden} t_2 + \left(1 - P_{garden}\right)\left[P_{suburb} \frac{d+10}{20} t_2 + \left(1 - P_{suburb}\right)\left(t_1 + t_2\right)\right] + t_3 + t_4 \qquad (11)$$

where $P_{garden} = P\left(garden, nest, true, 0\right)$, the true probability that a mature nest exists in the garden at time zero, and $P_{suburb} = P\left(suburb, nest, true, 0\right)$, the true probability that a mature nest exists in the suburb at time zero.

A suitable equation for estimating $P_{garden}$ and $P_{suburb}$ is

$$P_{garden} = P_{suburb} = \begin{cases} t_{\text{suburb}}/100, & t_{\text{suburb}} \le 100 \text{ years;} \\ 1 & t_{\text{suburb}} > 100 \text{ years.} \end{cases} \qquad (12)$$

where $t_{\text{suburb}}$ denotes the age of the suburb at time zero, the year in which the target house was built.

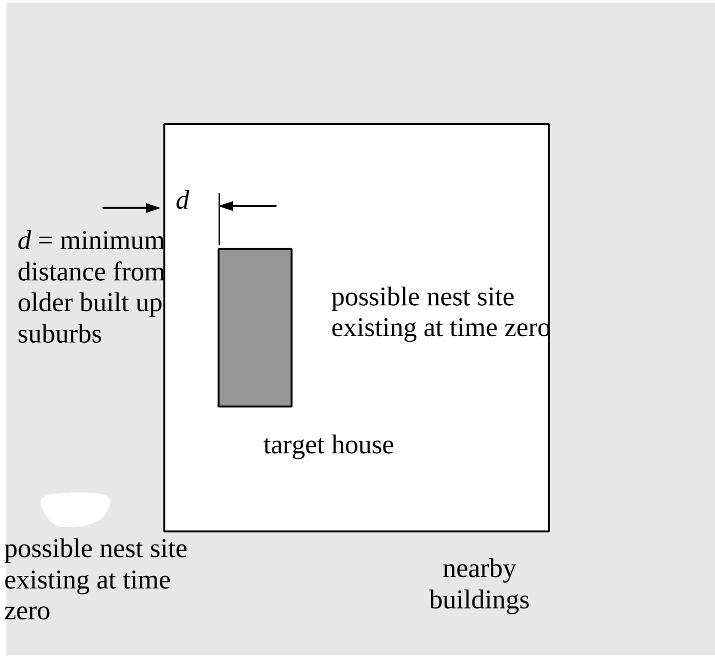


**Figure 5. House and land surrounded by existing buildings and nest sites at time zero.**

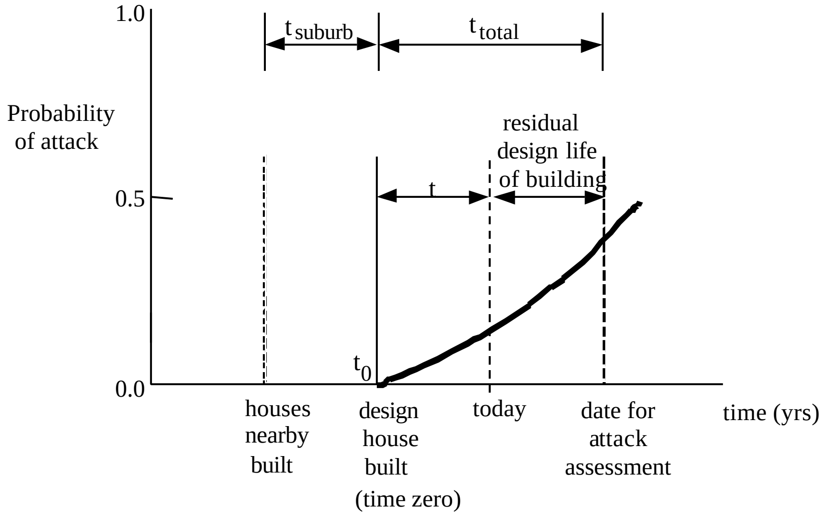


**Figure 6. Schematic illustration of event times.**

## 4 ZONATION FOR TERMITE HAZARD

The raw data from the Termite Tally was used to derive zones related to the termite hazard. To do this houses were grouped in terms of specified sets of parameters; then for each group, the termite hazard was taken to be proportional to the observed occurrence of termites in the garden. The data was first grouped to examine the effects of mean annual temperature and rainfall. For this purpose the data of the Termite Tally was grouped according to temperature-rainfall clusters, the cluster boundaries being chosen so that the sample size within each cluster is greater than 90.

The results are shown in Fig. 7. It is apparent that there is a reasonable effect of temperature on the termite hazard. However, the data does not show any clear effect of rainfall.

Therefore, a termite hazard map is proposed based on consideration of the mean annual temperature, denoted by $T_m$, as shown in Fig. 8(a), and consists of the three zones:

Zone 1: $T_m$ < 18°C;

Zone 2: 18 ≤ $T_m$ ≤ 25°C;

Zone 3: $T_m$ > 25°C.

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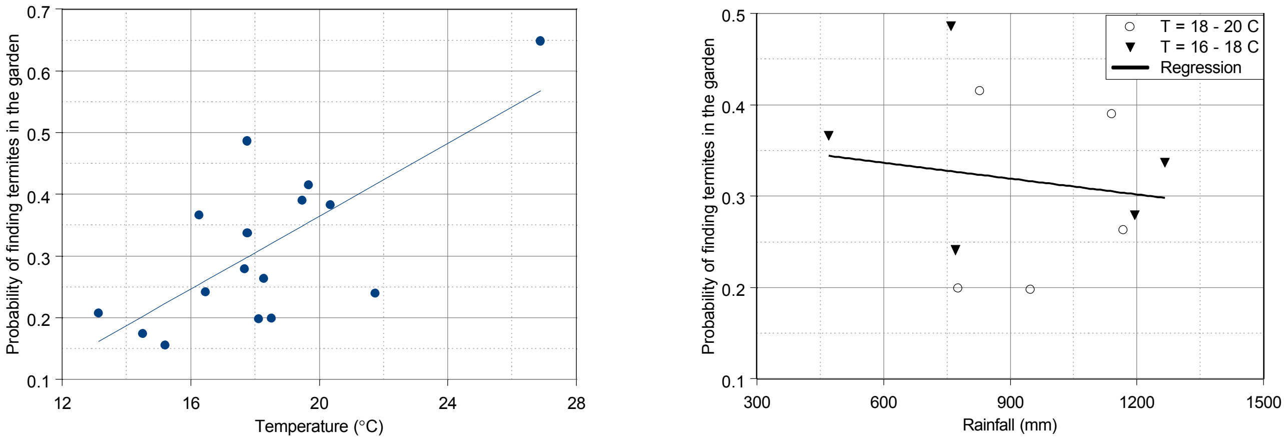
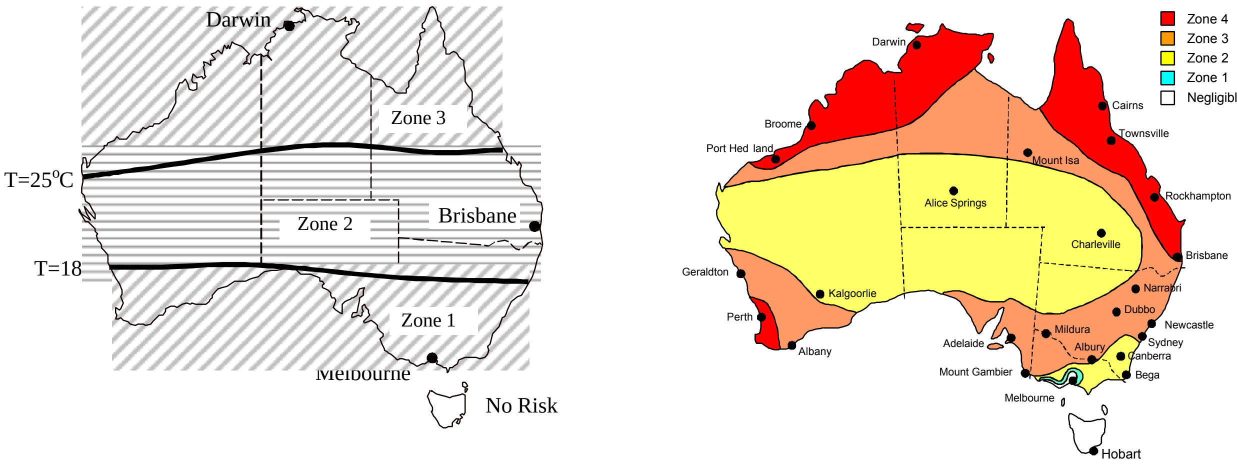
As an alternative, Cookson(1999) has proposed that the termite hazard be divided according to agro-ecological zones as shown in Fig. 8(b).




(a) Effect of temperature (b) Effect of rainfall

**Figure 7. Effect of temperature and rainfall on the apparent incidence of termites in the garden.**




(a) temperature (b) agro-ecological

**Figure 8. Termite hazard zonation.**

## 5 CALIBRATION WITH TERMITE TALLY DATA

The data from the Termite Tally was used to calibrate the probabilistic model of attack. For brevity, only the data expressed in terms of temperature zonation will be discussed in the following. The data used for calibration are shown in Figs. 9 and 10. The apparent probability of finding termites in the garden, denoted by $P(garden,termite,obsv,t)$, and the apparent probability that a house has been attacked at some time in the past, denoted by $P(house,attack,obsv,t)$. The data is plotted in terms of $t$, the age of the houses at the time of the survey for the Tally. Each plotted data point is based on data taken from the Termite Tally; the average sample size for each point is 165 with a minimum value of 51. There is obviously a very strong effect of age of house on the incidence of termite attack. In fact, it was found that in the data of the Termite Tally, the probability of termite attack on a house is more strongly correlated with the age of the house than with any other parameter.

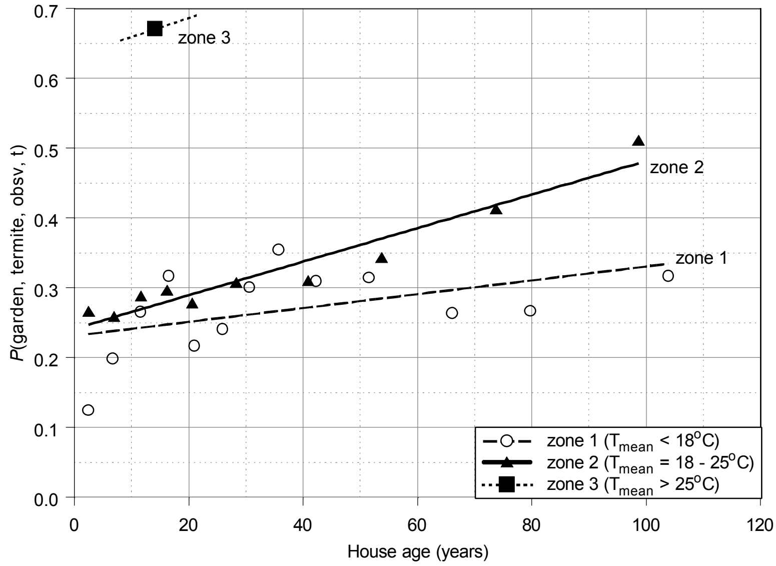



**Figure 9. Relationship between age of house and the apparent incidence of termites in the garden.**

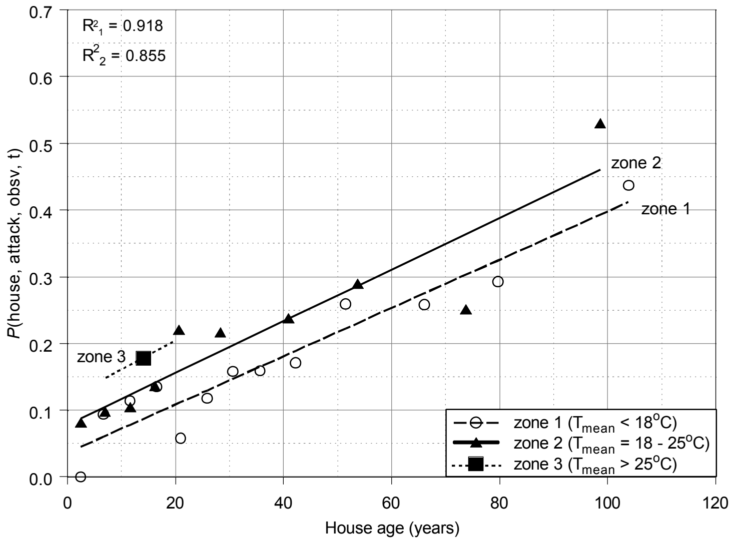



**Figure 10. Relationship between age of house and the apparent incidence of termites in the house.**

In applying the model, it was decided that in the absence of further information the effect of age of house would be taken to be the same in all zones. To do this a choice of the parameter $B = 0.004$ was used in Eq. (6). Thus to use the model to predict termite attack, only one input parameter is required and this was taken to be the mean value, denoted by mean($t$). Fig. 11 shows a fit of the model to data from the Termite Tally for average conditions.

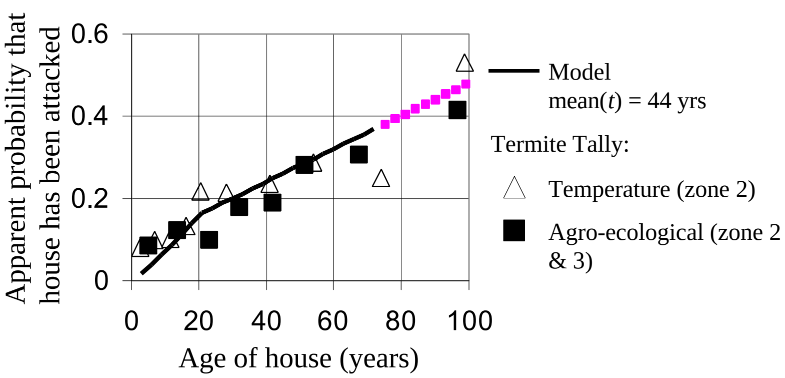



**Figure 11. Comparison of apparent risks derived from the model and the Termite Tally.**

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From the data of the Termite Tally as shown in Fig. 9, it is assumed for calibration purposes that a typical value of the apparent incidence of termites in a garden at time zero may be taken to be

$$P(garden, termite, obsv, 0) = 0.25 \quad (13)$$

If it is assumed that the reported frequency of termite occurrences represent only half the true value, and that the occurrence of termites in the garden is an indication that there is a nest there, then the true probability that a termite nest exists in a garden at time zero is

$$P_{garden} = P(garden, nest, true, 0) = 2 \times P(garden, termite, obsv, 0) = 0.5 \quad (14)$$

Furthermore it will be assumed that the probability of a termite nest occurring in the surrounding suburbs at time zero is the same as that of a nest occurring in the garden, then $P_{\text{suburb}} = P_{\text{garden}}$, and consequently

$$P_{\text{suburb}} = 0.5 \quad (15)$$

Substitution of these values and $d = 2$ into Eq. (11) leads to the mean attack time based on expert opinion, denoted by mean($t_{\text{expert}}$), to be given by

$$\text{mean}(t_{\text{expert}}) = 0.25\ \text{mean}(t_1) + 0.9\ \text{mean}(t_2) + \text{mean}(t_3) + \text{mean}(t_4) \quad (16)$$

To make use of this information, it is a reasonable approximation to assume that there exists a relationship

$$\text{mean}(t) = \beta\ \text{mean}(t_{\text{expert}}) (17)$$

where $\beta$ is a constant.

Table 5 shows a comparison between mean value estimates for average conditions. Two of the estimates are those by experts for average conditions, ie with $k_j = 0$ in Eq. (10). Two further estimates of mean values have been made by fitting the model to the Termite Tally data for average conditions. Assuming that the best expert opinion estimate for typical conditions in the past 20 years is mean($t_{\text{expert}}$) = 30 years and the best model fit to the Termite Tally data is given by mean($t$) = 45 years. Then an assumption that $\beta = 1.5$ is a reasonable estimate of the constant $\beta$ in equation. This value will be used in the application of the probabilistic model.

For completeness, estimates of related coefficients of variation are included with the data in Table 5; it is seen that for all practical purposes, the variability is the same for all the estimates.

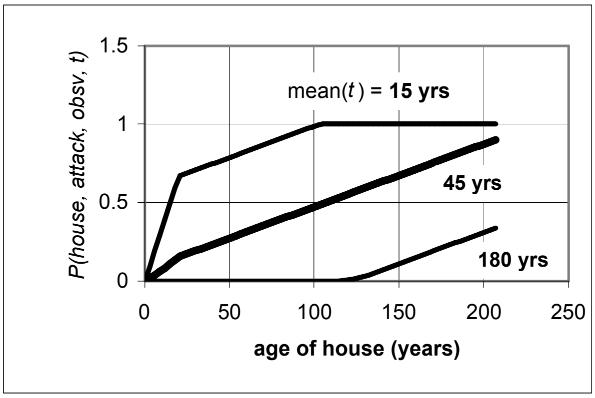
**Table 5. Attack time of average models**

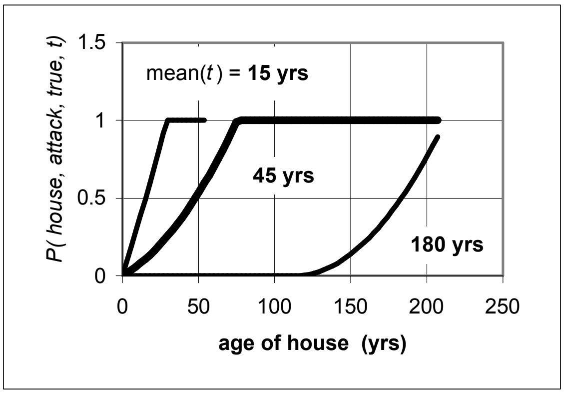
| *Data source* | *Mean time of attack (years)* | *Coefficient of variation* |
|---|---|---|
| Expert opinion model (no termite barrier present) | 23.8 | 0.42 |
| Expert opinion model (with termite barrier present) | 53.8 | 0.43 |
| Termite Tally (temperature zone 2) | 44.1 | 0.46 |
| Termite Tally (agro-ecological zones 2 & 3) | 50.2 | 0.43 |

## 6 APPLICATION OF THE PROPOSED MODEL

For any given set of input parameters, the value of mean($t_{\text{expert}}$) can be obtained from Eq. (16) of the Expert opinion model, Eq. (17) used to estimate the mean($t$), and then the probabilistic model used to give an estimate of the risk of termite attack for houses of various ages. Thus the model may be used to examine the cost effectiveness and relative risks involved with various termite resistance strategies.

It is of interest to see the predicted range of risks that are involved. By substituting $k_j$ = -1, 0, +1 into Eq. (10), the values of mean($t_{\text{expert}}$) are found to be 10, 30 and 120 years; from Eq. (17) this leads to values of 15, 45 and 180 years respectively for mean($t$). The resulting apparent and true risks for these three cases are shown in Fig. 12. The wide range of possible risks indicates that it is not realistic to group all houses in the same category. For example, Fig. 12(b) shows that the risk of attack on houses within the first 100 years may range all the way from 0 to 100%, depending on the input parameter conditions.




(a) apparent risk

(b) true risk

**Figure 12. Possible range of apparent and true risks of termite attack.**

## 7 CONCLUSIONS AND RECOMMENDATIONS

In this paper, the analysis of a Termite Tally and the collection of Expert Opinion are brought together in the form of a risk model of termite attack on timber housing. It represents the world's first predictive model of such an attack. The model takes into account the effects of numerous parameters, as listed in Table 1. In addition, the format for collecting Expert Opinion allows for the introduction of new parameters and concepts regarding termite attack as these become apparent.

A novel aspect of the model is that it introduces a parameter that represents the historical memory of an interviewee when providing data such as that reported in the Termite Tally. Application of the model show that the effects of the differences between the true and apparent risks of attack may not be significant for low risk scenarios such as in the design of house frames; however, there may be significant errors (on the unsafe side) in high risk scenarios, such as in the design of fencing and noise abatement structures, if the differences between true and apparent risks are not taken into account

The model may also be used to provide a probabilistic format for defining the performance specification of termite barriers and inspection procedures, and should prove to be an extremely useful tool in the development and assessment of asset management strategies and building regulations. Computer software based on the developed model has been written. It can be used for collecting further Expert Opinion and for making quantified risk predictions of termite attack.

Perhaps one surprising result observed in running the model is that it predicts that for the possible variety of scenarios in Australia, there is an extremely wide range of risks involved. Hence ignoring the application of this model and treating all scenarios as similar, can lead to very inefficient and/or very risky design procedures and building regulations.

## 8 ACKNOWLEDGEMENTS

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# Prediction Models For Engineered Durability Of Timber In Australia

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**Summary: This paper gives an overview of a major Australian project to develop an engineered approach to the design for durability of timber construction. The focus of this research has been to develop probability based prediction models for the effect of durability on the strength of timber structures. These models may be used for risk estimates of building performance. To date, models of attack by decay fungi, termites and corrosion agents have been developed. These models have been derived on the basis of expert opinion combined with data derived from tests on small clear specimens of wood and metal. The models are now undergoing calibration through the use of data from in-service structures. When calibrated, the models will be useful for the development of engineering design procedures, design optimisation and asset management strategies.**

**Keywords: durability, timber, structural, model, decay, termite, corrosion.**

## 1 INTRODUCTION

When a timber construction fails through structural collapse, the event is unexpected and the typical response is to sue the structural engineer concerned. However, should a timber construction fail because of a loss of durability, the general public perception is that this is a consequence to be expected in the use of wood. The reason for this difference in perceptions is that while structural designs are undertaken by the application of design codes based on predictive models of structural performance, design against durability is undertaken by a mixture of experience and adherence to good building practice. The expected performance in designs for durability is rarely stated.

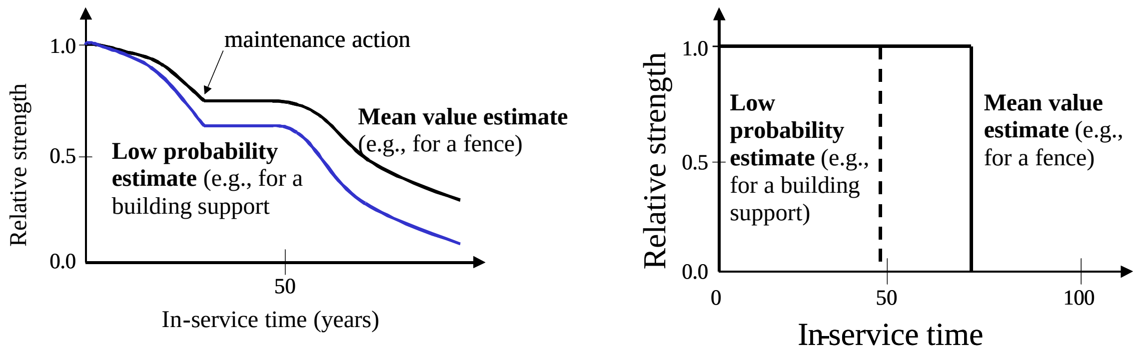
To remedy this situation, the Australian timber industry (through FWPRDC) has sponsored a large national project to develop engineered design procedures for timber construction. Progress to date is described in detail in a summary paper by Leicester (2001) and in several CSIRO laboratory reports (Leicester *et al.* 2001b, 2001c, 2001d and 2001e). The procedures developed are based on probabilistic prediction models. It is important to appreciate that in using probabilistic models, all outcomes can be stated in terms of risk, the single most significant measure of building performance. The models can include consideration of both the natural variability of relevant parameters and the uncertainty in our modelling concepts. Moreover the models can be formulated to include data from any source, including expectations originating from engineering experience. With the use of these probabilistic models, performance stated in risk terms can be used as criteria for

- engineering procedures
- cost-optimised design
- asset management strategies

The first and most important task of the project has been to develop probabilistic prediction models of the effects of various durability hazards on the performance of timber construction. Typical examples of the required model outputs are shown schematically in Fig.1. To date the durability hazards considered have been

- decay
- termite attack
- corrosion (of connectors)

In the future it is planned to include other hazards such as attack by marine borers, insects, and chemical and mechanical degradation.




**(a) Attack by decay fungi or corrosion agents** **(b) Attack by termites**

**Figure 1. Schematic illustration of the effects of durability hazards on the strength of timber construction.**

## 2 MODELS

### 2.1 Concepts

The basic components of the models developed are shown in Fig. 2. Here an attack model is used to predict the performance of a timber construction on the basis of a set of input parameters that define the problem. The most difficult part of the project has been to develop attack models in quantitative terms. Although there is extensive literature on hazard attack scenarios, very little of this is quantitative. Moreover it is obviously not possible to initiate experiments that last for more than about 3 years duration as part of this project. Accordingly it has been necessary to develop the format of models on the basis of limited experimental studies and information gleaned from the scientific literature, and then to calibrate these models on the basis of long-term quantitative data derived from in-service real constructions. In general the attack models require the following components to be defined:

- the attack pattern and/or attack scenario
- the rate of attack

### 2.2 Environment Factors

In order to quantify the effects of environment factors it is convenient to subdivide timber construction according to exposure conditions as follows:

- in-ground
- exposed
- protected

For protected conditions, the important locations of a house are

- sub-floor
- wall cavity
- roof space

These locations are illustrated schematically in Fig. 3. Figure 4 shows locations in Australia where the climate within the building envelopes of houses have been monitored.

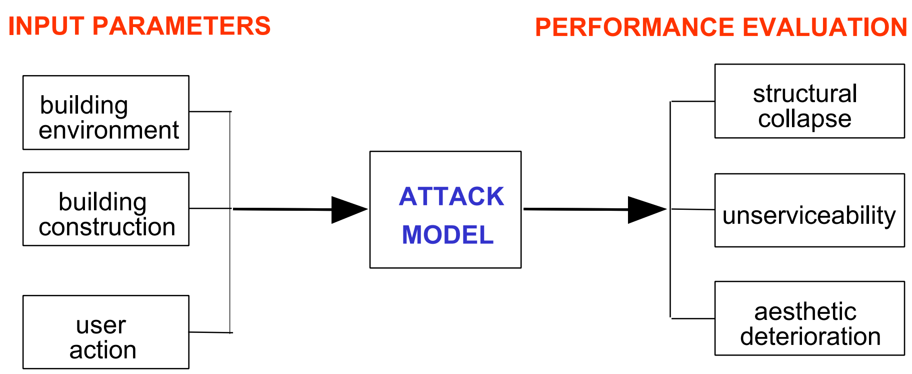



**Figure 2. Components of a probabilistic prediction model.**

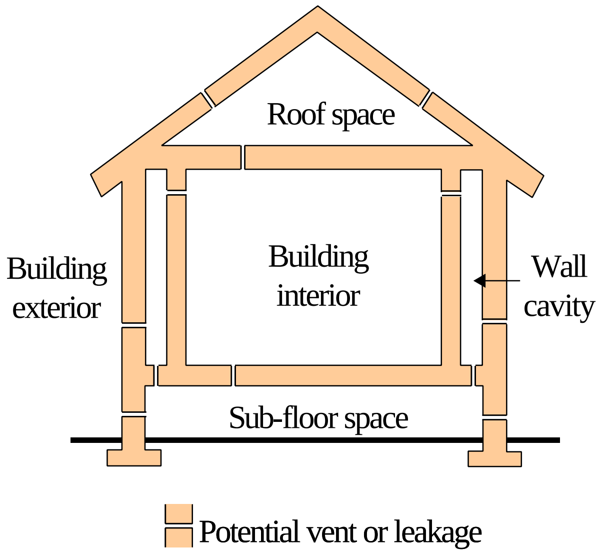



**Figure 3. Locations within the building envelope of a house.**


**Figure 4. Locations used for monitoring the climate within the building envelope of houses.**

Global environment parameters are derived by interpolating climate data from about 130 Meteorological Bureau stations scattered around Australia. This data, combined with local terrain and shelter effects, are used to derive local climate conditions for fully exposed construction. For exposed construction the effects of sheltering, such as illustrated in Fig. 5, have a significant effect on material degradation and must be evaluated quantitatively.

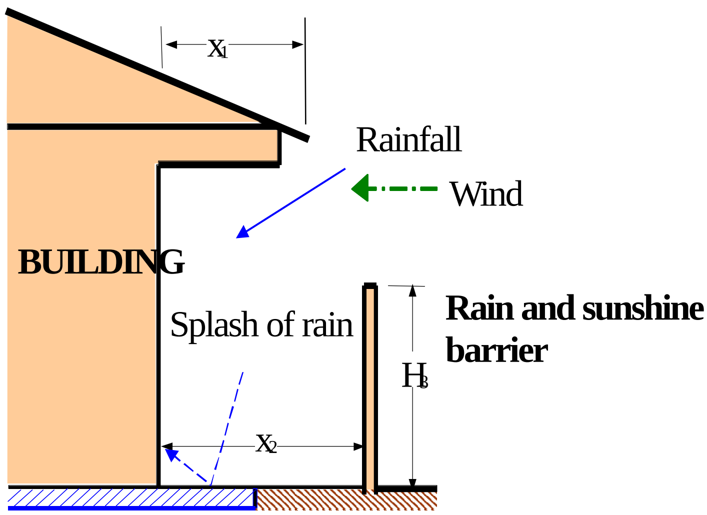



**Figure 5. Illustration of the effects of sheltering on the time of wetness of exposed construction.**

From the climate of exposed locations, the climate conditions within a building envelope of typical Australian housing may be derived through the use of empirical formulae such as (McGeachie *et al* 1999)

$$T_{cavity}(t) = 7.9 + 0.13\ T_{exterior}(t) + 0.41\ T_{exterior}(t\text{-}2) \quad (1)$$

$$H_{cavity}(t) = 28.4 + 0.24\ H_{exterior}(t) + 0.35\ H_{exterior}(t\text{-}2) \quad (2)$$

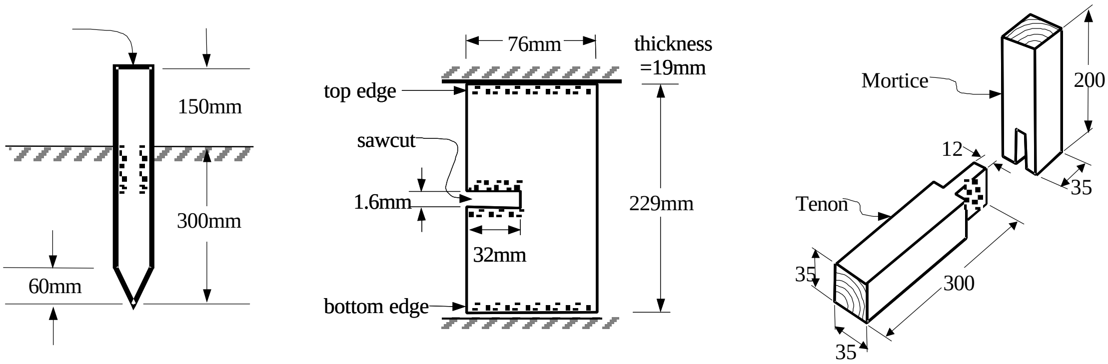
where $T$ denotes temperature (deg.C), $H$ denotes relative humidity (%), $t$ denotes the time (hrs), and the subscripts 'cavity' and 'exterior' denote conditions in the wall cavity and outside the house as shown in Fig. 3.

From temperature and humidity values, most of the environment parameters that affect durability can be derived. In addition to this, it is also necessary to have procedures for estimating the concentration of salt and other pollutants in the air.

### 2.3 Decay

The models for decay are based on data obtained from tests described elsewhere (Beesley 1978, Creffield *et al.* 1992, Thornton *et al*, 1994 Cause 1993) on small clear wood specimens of the types shown in Fig. 6. The locations at which these specimens were tested are shown in Fig. 7. The in-ground specimens were monitored for about 30 years and the exposed above-ground specimens for about 10 years. Most of the specimens were from outer heartwood timber. All told the studies involved about 80 species and 5 preservatives, although not all species or preservatives were used for every test. In addition, the effects of several maintenance treatments were assessed from a study of 60 in-ground poles located at the Wedding Bells test site near Coffs Harbour in New South Wales (Gardner *et al.* 1998); these poles had been under test for 15-25 years.

From the data, idealised but quantified models for the rate of decay, as shown in Fig. 8, were derived. The strength capacity of a structural element is then obtained using the assumption that decayed wood has no strength, while undecayed wood has its full initial strength. This assumption is still under investigation. Models of decay rates were derived for corewood, outer heartwood and treated and untreated sapwood as shown schematically in Fig. 9. These models were then combined with an assumed attack scenario and attack pattern to estimate the effective residual structural cross-section and hence load capacity with time. Examples of assumed attack patterns are shown in Fig. 10.




(a) In-ground stake (b) Exposed climate index panel (c) Exposed L-joint

**Figure 6. Small clear wood specimens used to develop the models for decay.**

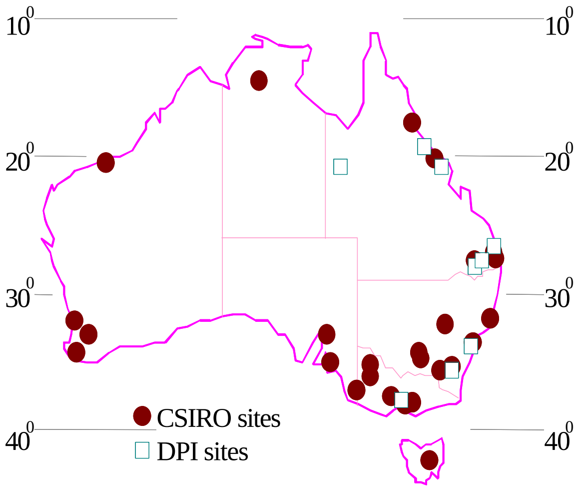



**Figure 7. Locations of test sites used to measure decay**

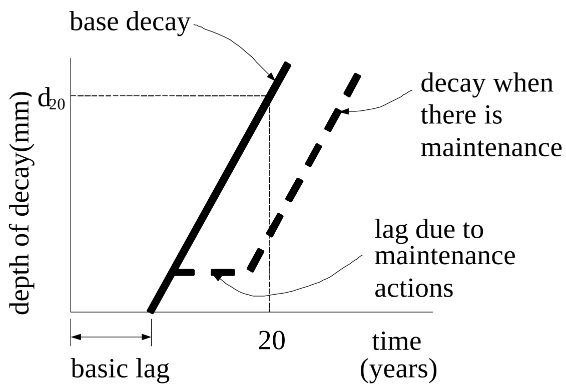



**Figure 8. Idealised model for attack by decay**

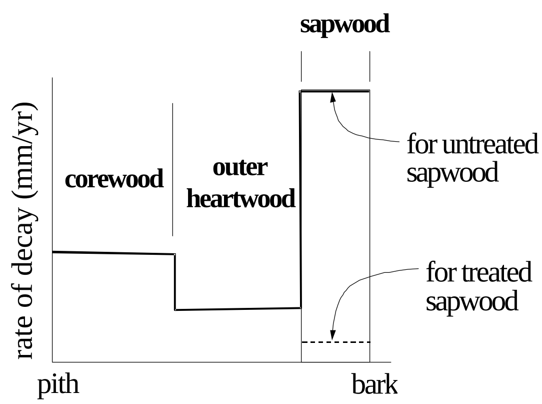



**Figure 9. Schematic illustration for the relative rates of decay within a pole**

From the field data from small clear wood specimens, the effect of climate on in-ground decay was found to be reasonably well related to an index that is a function of the following parameters:

- annual rainfall
- number of months per year that have a rainfall less than 5mm
- mean annual temperature

Similarly a climate index for the decay of exposed timber was found to be related to the following parameters:

- mean annual temperature
- annual time-of wetness
- mean annual vapour pressure deficit

The computed climate zones for in-ground and exposed timber in Australia are shown in Fig. 11. No direct measurements have been made for the case of decay in a protected environment. However from microbiological considerations it would appear that decay cannot initiate without the effects of wetting by free moisture; i.e. high humidity alone is not enough to commence decay (Zabel and Morell 1992). Within a building envelope, free moisture may arise from rainwater ingress, leakage from domestic sources or condensation. In the absence of other information, the decay equations developed for exposed construction would appear to be a useful starting point.

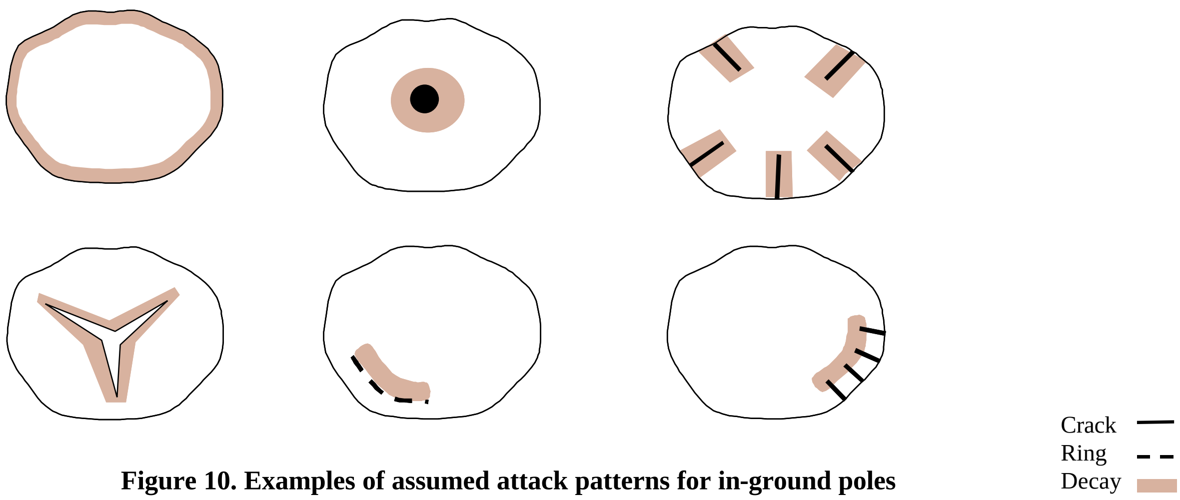



**Figure 10. Examples of assumed attack patterns for in-ground poles**

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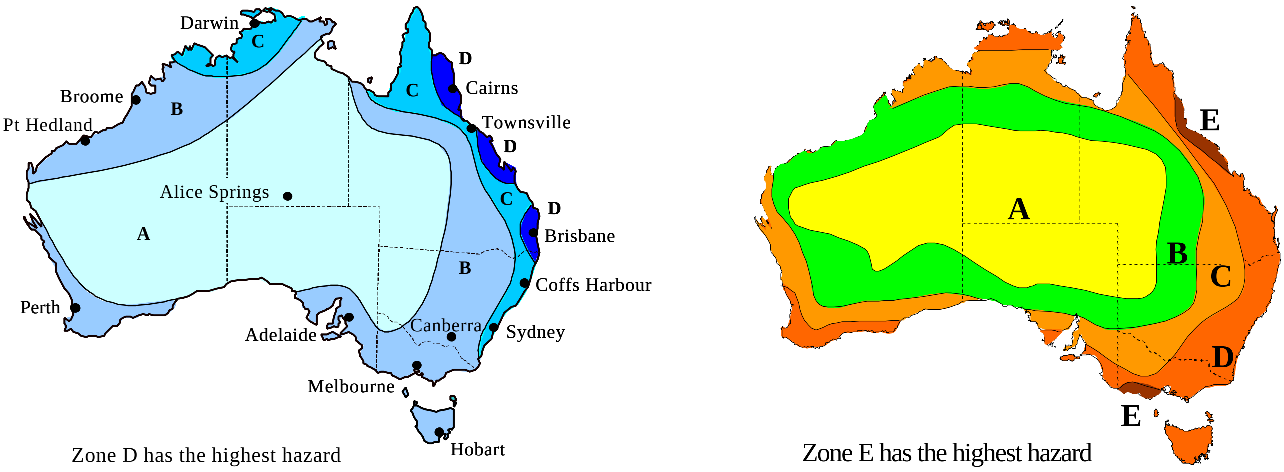



(a) In-ground timber (b) Exposed timber

**Figure 11. Climate zones for fungal decay of timber.**

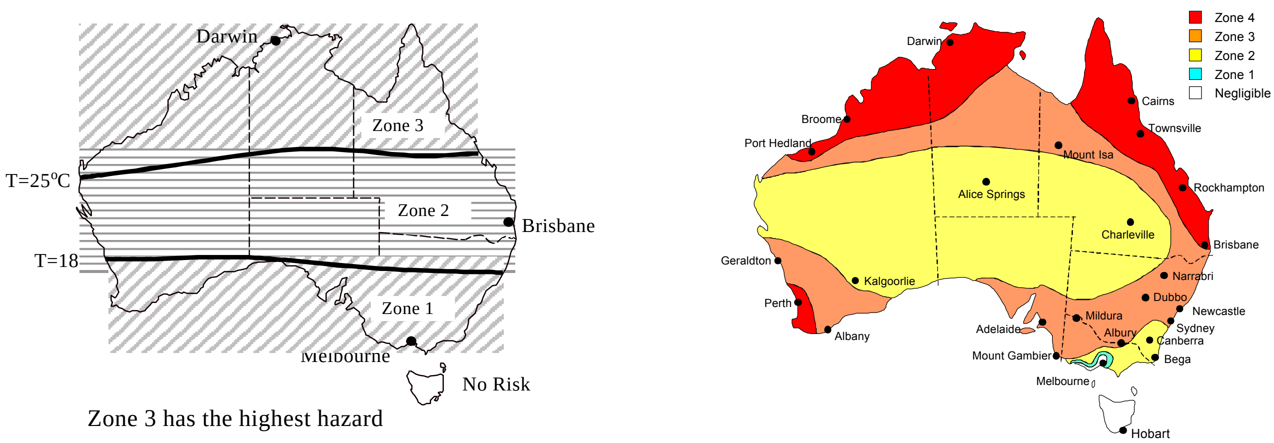
## 2.4 Termite Attack on Houses

The model for termites was developed initially as a model to reflect the opinions of experts such as researchers and pest controllers (Leicester and Wang 2001, Leicester *et al* 2001d). This model takes into account the following parameters in predicting the time to attack a house;

- mean annual temperature
- age of suburbs
- building construction
- type of termite barrier, if any
- inspection frequency
- number of potential nest sites
- food sources
- soil type

From this model an average estimate of the time to attack destructively is derived. This mean estimate is then used in a probabilistic model that predicts the probability of attack depending on the age of the house. The model has been calibrated with data obtained from a termite survey of 5000 houses chosen at random around Australia (Cookson 1999). One important aspect in using the data from this survey was to assume that the memory of the occupants surveyed was limited to events that occurred no earlier than 20 years before the survey date. This observed risk will be termed the "apparent risk'.

Using the data from the housing survey, it was found possible to relate the termite hazard either to agro-ecological zones or to temperature zones. The derived hazard zones are shown in Fig. 13. Figure 14 shows the good agreement obtained between the model predictions and the observed apparent risk of attack for average conditions. Figure 15 shows that the possible range in the risk predicted by the model is extremely great, depending on the choice of input parameters used, and cannot be ignored in practical applications.




(a) Temperature zonation (b) Agro-ecological zonation

**Figure 13. Climate zones for termite attack.**

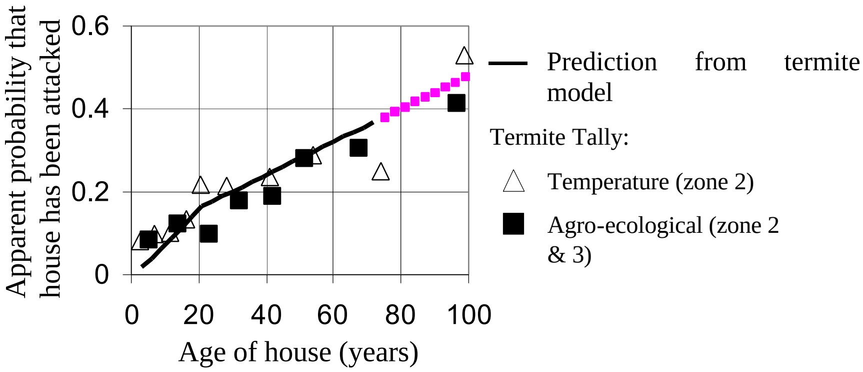



**Figure 14. The apparent risk of attack for average conditions.**

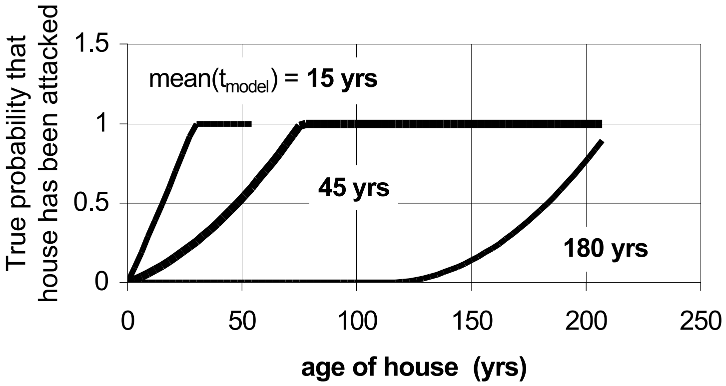



**Figure 15. The range of predicted true risks for Australia.**

### 2.5 Corrosion of Metal Fasteners

The model for corrosion of metal fasteners is focussed on estimating the loss of metal with time. Once this loss is known, the effect on the strength capacity of a connector can be calculated from basic mechanics concepts. It is assumed that metal lost is defined in terms of a depth of corrosion depth $c$ given by

$$c = c_o t^n \tag{3}$$

where $c_o$ is a parameter that depends on the type of metal and the environment conditions, and $t$ denotes the in-service time. The two types of metal that have been investigated to date are bright steel and zinc coating.

For the case of metal embedded in wood, the relevant parameters in the model are

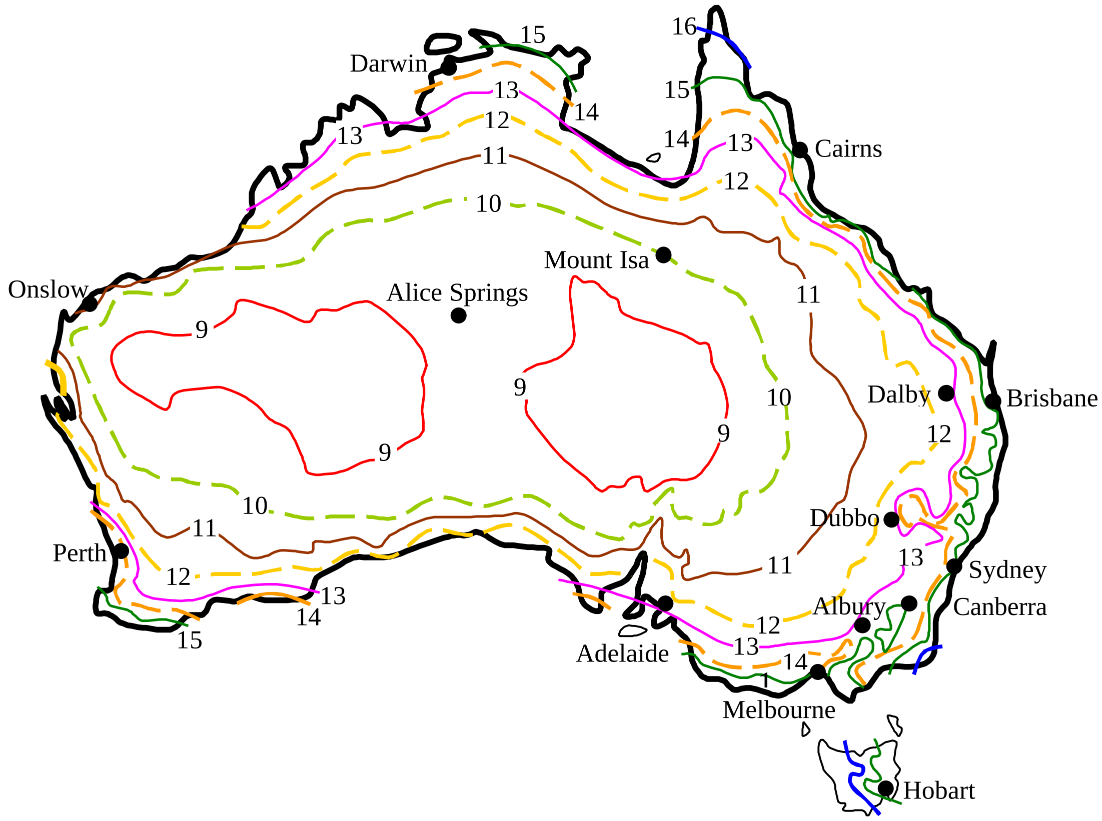
- the timber substrate (including the type of preservative, if any)
- the moisture content of the timber.

The moisture content of the timber depends on the location of the connector and it is loosely related to the mean annual moisture content of wood in outdoor sheltered conditions. The distribution within Australia of this moisture parameter is shown in Fig. 16.

For the case of metal exposed to air, such as occurs with the plate of toothed-plate connectors, the relevant corrosion parameters are

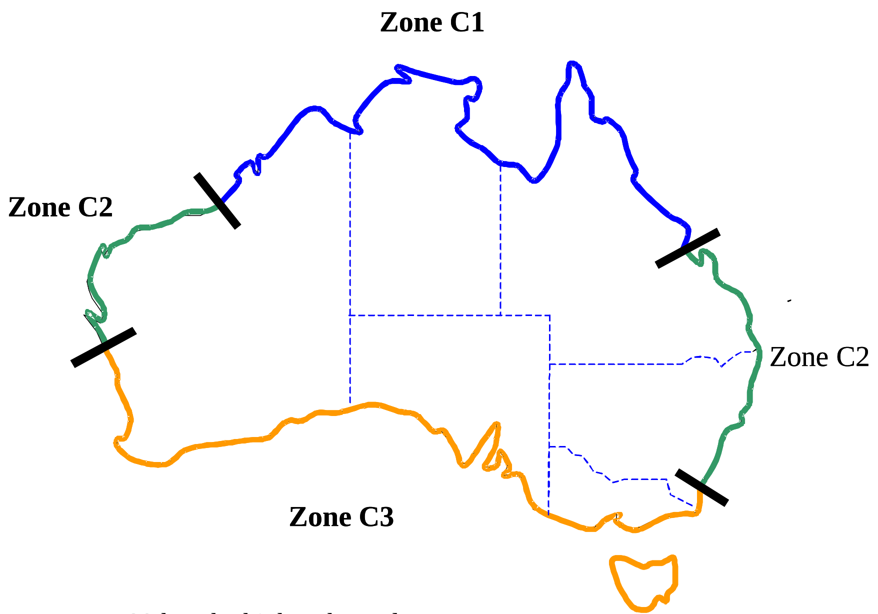
- the salt or pollution concentration in the air
- time of metal surface dampness.

Pollutant concentration depends on the type of pollutant and the distance of the construction from the pollutant source. Salt concentration depends on the distance from the sea coast and the sea-state activity near that coast (Cole *et al.* 1999, Leicester *et al.* 2001c). The coastal zonation of the Australian coastline for this purpose is shown in Fig. 17.




Higher moisture contents indicate higher hazards

**Figure 16. Mean annual moisture content of wood in sheltered conditions (per cent).**




Zone C3 has the highest hazard

**Figure 17. Coastal zones for computing the corrosion of exposed connector surfaces.**

## 3 MODEL CALIBRATION

As indicated in the Introduction, the basis for the development of models has been based largely on a mixture of data obtained from tests on small clear pieces of wood and/or expert opinion. However before the models are used for practical applications, they must first be calibrated with data obtained from in-service construction. This data is essential to derive and/or verify assumed attack patterns and long duration effects. Thus the full range of data to be considered are derived from the following sources:

- expert opinion
- laboratory tests on small wood or metal specimens
- field tests on small wood or metal specimens
- field tests on full size constructions
- in-service measurements on real construction

Currently, a major effort is in progress to collect in-service data. Table 1 gives a summary of the data that has been obtained to date.

**Table 1. Summary of project data samples**

| *Hazard type* | *Number of data items* | | | |
|---|---|---|---|---|
| | *Lab studies* | *Field studies* | | *In-service studies* |
| | *Small specimens* | *Small specimens* | *Full size specimens* | |
| In-ground decay | — | 5000 | 150* | 200 |
| Exposed decay | — | 4000 | — | 1500 |
| Termite attack | — | — | — | 5000 |
| Corrosion | 200 | 700 | 300** | 20 |

* Poles at Wedding Bells site.

** Gang-nail plates.

## 4 CONCLUDING COMMENT

At this time useful working models and associated software have been derived for predicting the structural performance of timber construction subjected to attack by decay fungi, termites and corrosion agents. However the model of attack within a building envelope by decay fungi needs further investigation. Calibration of these models with data from in-service construction is in progress.

Future work will focus on the use of these models for engineering design and asset management purposes. Models for attack by other hazards will be developed. The scope of the models will be extended to include the prediction of non-structural effects such as for example aesthetic and serviceability considerations.

## 5 ACKNOWLEDGEMENTS

The authors are indebted to the FWPRDC (Forestry and Wood Products Research and Development Corporation) for funding this project and also for the contribution of other members of this project team including John Thornton, Gary Johnson, Dave Gardner, Myron Cause, Laurie Cookson and Craig Seath.

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# Engineering Design Methods for Service Life Prediction

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**Summary: Service Life prediction is required when designing large infrastructure projects and is one of the objectives to be covered according to the European Construction Products Directive. It is of major concern for the owners of any kind of structure or asset.**

**Large investment bodies (transport ministries, property funds, universities, consulting companies, etc.) collect data on service life for specific components. The need arises however; to set up generally applicable methods using basic data and adapting these to e.g. different application, exposure and user conditions.**

**Today, two main methods can be distinguished:**

**1. For large infrastructure projects, teams of specialists are set up, investigating service life under the exact conditions envisaged (Great Belt, Western Scheldt Tunnel, etc.) and developing tailor made solutions, quite often using probabilistic methods.**

**2. In ISO 15686 – 1:2000 the so called "factor method" is proposed where seven factors are applied to the basic value of service life catering for the individual quality, exposure and in use condition of the building component considered.**

**These two methods exhibit heavy drawbacks. The probabilistic methods and most other tailor made techniques applied are mostly based on probabilistics and are too elaborate to be used on standard applications such as office buildings or ordinary road bridges. The factor method on the other hand is fairly simple, but identifies the main parameters influencing service life. The result however, is only a single figure for service life and does not take into account at all the variability of the processes involved.**

**Thus within the CIB/RILEM Working Commission 175-SLM: "Service Life Methodologies", three subtask groups were set up under the heading "Performance based methods of service life prediction", one of them with the goal of setting up generally applicable "Engineering Design Methods" fitting in between the two methods described above. This was thought to be done either by simplifying scientific models or by expanding the factor method towards the more sophisticated models.**

**The subtask groups have produced state of the art reports to be published in 2002 as a basis document for further developments. As engineering design method, a principle solution is proposed in this paper which can be applied to the factorial method for standard cases as well as to other set-ups employing mathematical relations for service life. This is achieved by not using plain factors but only, probability density functions instead. These are established using reliable and understandable engineering techniques. Three Examples are shown to illustrate the proposed procedure.**

**Keywords. Service life, engineering design method, factor method, probabilistic method, ISO 15686**

## 1 INTRODUCTION

This paper reports on work done with regard to the service life prediction under the wings of the CIB/RILEM Working Commission 175: "Service Life Methodologies", in close co-operation with ISO TC 59 / SC 14 " Building Construction - Design Life". Three subtask-groups were established in 1999 under the heading "Performance based methods of service life prediction". For one subtask-group the goal was set to developing generally applicable "Engineering Design Methods (EDM)" for service life design, methodically fitting in between the probabilistic and the factorial methods. This was thought to be done either by simplifying scientific models or by expanding the factor method towards the more sophisticated models. The three subtask-groups compiled state of the art reports as a basis for future development (CIB/RILEM 2002).

## 2 INCENTIVES FOR SERVICE LIFE PREDICTION METHODS

Service life is of major concern for the owner of any kind of structure or asset. Its prediction is therefore required by investors, be it a state organisation or a private body, when designing large infrastructure projects, and for private investors holding large property assets. Service life is as main parameter involved in all economic considerations as return of the invested capital, investment planning for maintenance and refurbishment, etc.

Service life is also the essential requirement to be covered according to the European Construction Products Directive: Within the period of service life all main qualities of a constructed asset or product have to be met, viz. the work has to be mechanically safe, protected against failure as a consequence of fire, it has to be hygienic, safe to use, with an acceptable noise level, and has to maintain sufficient comfort to the user at a low energy consumption (ECC 1988, Appendix 1).

## 3 STATE OF THE ART

### 3.1 Relevant data available

Large investment bodies (transport ministries, property funds, universities, consulting companies, etc.) collect data on service life for specific components. For large populations of similar structures, service life planning in this case can be a relatively simple matter of applying the respective collected data. In this case no elaborate models are required, as the data fit more ore less closely to the problem under consideration.

### 3.2 Lack of relevant data

In most cases, directly applicable data are not available. This is namely the case, when the works are

- not reasonably comparable, or
- exposure conditions are different, e.g. due to sites in other climatic environments, or
- they are built in areas where no sufficient service life data are available, or
- new products have to be applied in new climatic conditions based on tests under other conditions.

Therefore, the need arises, to set up generally applicable methods using basic data and adapting these to e.g. different application, exposure conditions and user conditions, etc.

Today, two main methods of service life prediction can be distinguished: the probabilistic methods and the factorial method.

### 3.3 Probabilistic methods

For large infrastructure projects, teams of specialists are set up, investigating service life under the exact conditions envisaged (Great Belt, Western Scheldt Tunnel, etc.). They develop tailor made solutions, most often using quite elaborate probabilistic methods (see literature, viz. DuraCrete Design Manual, Edvardsen & Mohr 2000, Breitenbüchner et al, Fagerlund, Leira, Lounis et al, Siemes, all in 1999, etc.).

These techniques are out of the reach of the ordinary planner, at least as long as he is not in the position to acquire the inherent knowledge easily and quickly, and as long as there are no easy-to-use design aids available, properly introduced to the engineering community.

### 3.4 Factorial method

ISO TC 59/SC 14 " Building Construction - Design Life" has set up six working groups so far successfully producing several parts of ISO 15686 "Buildings and constructed assets: Service Life Planning". In the first part, the so called "Factor method" is proposed applying seven factors to the basic value of reference service life, catering for the individual quality, exposure and in use condition of the building component considered (ISO 15686-1, 2000, for working examples see annexes E & F).

This method, applied plainly as set up in the code, yields one single value for the average service life. A customer however, is not only interested in the average value; he has to know as from when substantial renovations or replacements have to be expected. This usually is the case long before the average service life is reached.

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### 3.5 Appraisal of the state of the art

These two methods shortly described above exhibit to heavy drawbacks:

- The probabilistic methods and most other tailor made techniques which are mostly based on the theory of probabilistics are too specialised, too cumbersome or too complicated to be used by ordinary planners on standard applications such as office buildings or ordinary road bridges.
- The factor method requires the estimation or evaluation of up to seven individual factors and nevertheless gives a single figure for service life only. It does not at all take into account at all the variability of the processes involved in the ageing of the structure or component involved.

### 3.6 Performance based methods of service life prediction

Within the CIB/RILEM Working Commission 175: "Service Life Methodologies", three subtask groups were set up under the heading "Performance based methods of service life prediction". The three groups are titled as follows:

- Scientific (probability, stochastic) methods,
- Engineering design methods and
- Factorial method.

The second group with the goal of setting up generally applicable "Engineering Design Methods (EDM)" fits in between the two other methods already described above. EDMs were thought to be gained by either by expanding the factor method towards the more sophisticated models or by simplifying scientific models for general use.

Firstly, the three subtask-groups have produced state of the art reports being put together as a basis document for further developments (CIB/RILEM 2002).

## 4 ENGINEERING DESIGN METHODS

As a first engineering design method, a principle solution is proposed which can be applied to the factorial method for standard cases as well as to other set-ups employing mathematical relations for service life. This is achieved by not using plain factors but probability density functions instead. These are established using reliable and understandable engineering techniques.

### 4.1 Data acquisition: the recursive Delphi method

In many cases, data are not available for ready use in the form of values for the main factors of influence defined in the equation for service life, let alone in the form of distributions. A very valid method is the so-called (recursive) Delphi method. (The author has been engaged in this type of data acquisition as far back as the 1980’s in the field of (industrial) risk engineering.)

1. In a first step, a panel of experts is called together and asked on their professional opinion on the distributions of the different factors, their type of distribution (normal, lognormal, Gumbel, etc), their mean values, standard deviations.
   Usually, it is easier to define fractiles based on experience and professional judgement, say 5 or 10%, mean values and 90 or 95%. Experts can quite well define these values, when asked precisely.
2. The second step involves the service life calculation using the expert panel’s input. The distributions are used instead of plain factors in the mathematical formulations for service life.
3. The third step is a thorough discussion of the results and of the dominating parameters. Sensitivity analysis has proved to be an important tool at this stage. Very often, as a consequence of this appraisal, the data or the models have to be adjusted to yield results judged reasonable in those areas of the problem where the experts have sufficient practical experience. After this fine-tuning of the model and the density distributions of the factors, the general problem can be tackled successfully.

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**Figure 1: Recursive Delphi method**

### 4.2 Application of Engineering Design Methods

The general engineering design method (EDM), at this stage, is defined as “any simple mathematical relation (as simple as possible, but not too simple) worked on, using distributions of any kind for the individual factors in the relation”. This procedure yields as result distributions for the expected service lives, information, which can easily be understood and interpreted by the decision-makers.

The following examples show three variations of the engineering design method EDM:

The first example uses all seven factors of ISO 15686-1, 2000 under the assumption, that the information for defining the respective distributions is readily available.

The second example is working on limited information only. The equation has to be modified, and the respective distributions are set up indirectly, partially from information on the resultant differences of service life.

The third example bases on a completely different equation, which is established using common engineering sense. It is normalised using an average result calculated on the basis of an equation based on an error-function.

## 5 EXAMPLE 1: EXPANSION OF THE FACTORIAL METHOD

This example demonstrates the basic procedure with the following full equation (1). All factors are applied as indicated, using four different distributions in order to demonstrate the ease of application of this expanded factorial method. The example is based on the worked example for softwood windows given in ISO/CD 15686-1 (1997) but the factors are indexed as in the edition 2000 of the code:

$$PSLDC = RSLC \cdot f_A \cdot f_B \cdot f_C \cdot f_D \cdot f_E \cdot f_F \cdot f_G \qquad (1)$$

*PSLDC* is the predicted service life distribution of the component based on the reference service life *RSLC*. The factors indices are: *A* for the quality of the component, *B* for the design level, *C* for the work execution level, *D* for the indoor environment, *E* for outdoor environment, *F* for in-use condition and *G* for maintenance level (see Tab. 1).

### 5.1 Estimated service lives for the windows in all four faces

The basis of the numerical example is a squared building of a length of 50 m, a width of 25 m and a height of 30 m. The long sides are facing due south and north respectively. The windows of the four façades of the building are in the example treated separately. Table 1 shows the assumed relevant conditions for all factors and faces. Therefrom the factors for the three fractiles 5%, 50% and 95% are defined in the sense of the Delphi method, in this case based on the factors and their description given in ISO/CD 15686-1:2000.

**Table 1: Fractile values of factors**

| Factor | Face | Relevant conditions | Factors for the fractiles 5% / 50% / 95% |
|---|---|---|---|
| $f_A$ Quality of component | all | general variations of components | 1.2 / 1.5 / 1.8 |
| $f_B$ Design level | all | good, identical value | 1.2 |
| $f_C$ Work execution level | all | general variation, but insufficient quality repaired | 1.0 / 1.2 / 1.5 |
| $f_D$ Indoor environment | S | occasional risk of condensation | 0.9 / 1.0 / 1.2 |
| | W | medium risk of condensation | 0.8 / 0.9 / 1.1 |
| | N | high risk of condensation | 0.7 / 0.8 / 0.95 |
| | E | medium risk of condensation | 0.8 / 0.9 / 1.1 |
| $f_E$ Outdoor environment | S | occasional cycling dry / damp | 0.8 / 1.0 / 1.3 |
| | W | regular cycling dry / damp | 0.6 / 0.8 / 1.0 |
| | N | sheltered from rain | 1.0 / 1.2 / 1.5 |
| | E | occasional cycling dry / damp | 0.8 / 1.0 / 1.3 |
| $f_F$ In use conditions | S | occasional access by children [1]) | 0.8 / 1.0 / 1.2 |
| | W | regular access by children [1]) | 0.6 / 0.8 / 1.0 |
| | N | occ. / reg. access by children [1]) | 0.7 / 0.9 / 1.1 |
| | E | occasional access by children [1]) | 0.8 / 1.0 / 1.2 |
| $f_G$ Maintenance level | all | painted on judgement of caretaker | 0.9 / 1.0 / 1.1 |

Note: [1]) according to example in ISO/CD 15868-1 (1997), other descriptions for wear and tear may appear more realistic.

The values for the fractiles given in table 1 are approximated by the density functions given in Table 2 for the ease of processing. The functions chosen represent those generally used: deterministic, normal, lognormal and Gumbel (extreme-value) distributions. The program used for the data processing, VaP 1.6 (1996), supports 11 types of distributions and user defined functions. It only requires 2.2 MB of disc-space in the expanded state.

The factors for *C* reflect the fact, that a quality of workmanship usually is such, that all parts not being sufficient are upgraded to meet the requirements, whereas those exceeding the requirements are naturally left at their higher level. This procedure leads typically to asymmetric distributions (in this example approximated using an extreme value “Gumbel” distribution) with very few components below the satisfactory level of 1.0 and a fairly wide spread upper area of the distribution.

The variables *m* and *s* are the first and second moments of the respective distributions (see VaP 1.6 1996).

**Table 2: Resultant predicted service life distribution of the components (*PSLDC*)**

| Factor | Type of Distribution | Face | | | |
|---|---|---|---|---|---|
| | | South m / s | West m / s | North m / s | East m / s |
| RSLC | Deterministic | 25 years | 25 years | 25 years | 25 years |
| $f_A$ | Normal | 1.5 / 0.185 | 1.5 / 0.185 | 1.5 / 0.185 | 1.5 / 0.185 |
| $f_B$ | Deterministic | 1.20 | 1.20 | 1.20 | 1.20 |
| $f_C$ | Gumbel | 1.25 / 0.10 | 1.25 / 0.10 | 1.25 / 0.10 | 1.25 / 0.10 |
| $f_D$ | Lognormal | 1.05 / 0.10 | 0.95 / 0.10 | 0.80 / 0.10 | 0.95 / 0.10 |
| $f_E$ | Lognormal | 1.05 / 0.20 | 0.80 / 0.20 | 1.25 / 0.20 | 1.05 / 0.20 |
| $f_F$ | Normal | 1.0 / 0.12 | 0.80 / 0.12 | 0.90 / 0.12 | 1.0 / 0.12 |
| $f_G$ | Normal | 1.0 / 0.06 | 1.0 / 0.06 | 1.0 / 0.06 | 1.0 / 0.06 |
| PSLDC (years) | Lognormal [1]) | 62.0 / 20.4 | 34.2 / 11.8 | 50.6 / 14.8 | 56.1 / 18.6 |

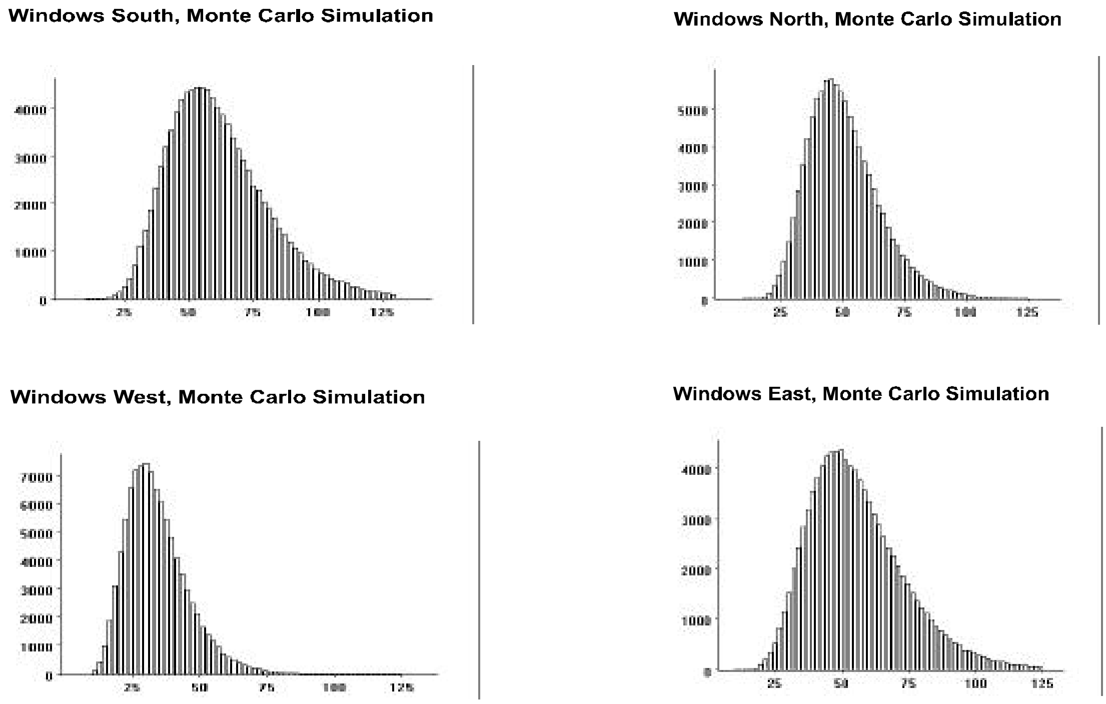
Note: [1]) close fit

The results were calculated by direct methods using VaP 1.6 (1996) and are shown on the last line of Table 2. The average results of two runs of a Monte Carlo simulation match the mathematically calculated results by a maximum difference on the average value of 0.1 years. These simulations yielded detail results as graphically shown for each facade as in Fig. 2. Dividing the relative densities shown on the vertical axis through the number of runs, in this case by 100’000, derives at the absolute densities.

### 5.2 Comparison of the four façades

The results for the estimated service lives of the four façades in Fig. 2 are different with respect to several aspects. First one notices the different widths of the distributions, in accordance to the values (second moments) in Fig.2). The spread is largest for the south face and narrowest for the west face. Some of this effect is relative: Due to the higher average value, the same relative spread is larger in years.

The west face shows the shortest service life, as expected, mainly due to the unfavourable outdoor climate and the in-use conditions. The effect of the higher risk of condensation, assumed for the north face indoor climate, is offset by the more favourable outdoor climate. The main difference originates from the in-use conditions. Both effects combined yield some 15% less estimated service life.




Note: Densities are the result of $10^5$ runs of a Monte Carlo simulation

**Figure 2: Distributions of predicted service lives PSLDC for all four facades**

### 5.3 Financial demand

For the planning of the maintenance funds, the functions for the service lives of the similar building parts can be superimposed. In general, this has to be done for all parts of a building considered. For the superposition, costs have to be allocated to the different groups of building parts.

In this example, the superposition of all window areas to be replaced is executed only, in order to be able to show typical results. (It is assumed for this purpose, that the windows cover 40% of the area of the respective façades.)

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**Figure 3: Financial demand**

The superposition yields an asymmetric function having a steep increase up to a peak demand of replacements of 48 $m^2$/year after 37 years (see Fig. 3). Then the demand decreases at a gentle slope down to 10 $m^2$/year after 70 years.

In a next step, the same service life functions can again be applied to the replaced windows, and the results of the multiple replacements are summed up, leading to a fairly constant replacement function. These steps are omitted here for clarity.

In general, the financial demand for similar parts tends to merge into a one-peak function. The superposition of the functions of all different parts of a building is more likely to result in several peaks or even a relatively steady demand over the lifetime of the building considered, starting at a certain age of the building.

## 6 EXAMPLE 2: MODIFICATION OF THE FACTORIAL METHOD

This example deals with service life of fibre cement slates used as wall cladding. The input data is fairly scarce, far form being complete and not directly suited for application in service life calculation. The basis of this calculation is the factorial method as set up in ISO 15686-1:2000, modified to suit this specific case. The example shows that interpretation of available limited data can nevertheless lead to a coherent and satisfactory service life prediction.

### 6.1 Available data

A manufacturer supplied data from his experience as follows:

- The quality of production of the slates can be derived from the bending strength, assumed to be characteristic value for the mechanical strength. The mean value $m$ lies 20% above the strength required and the standard deviation $s$ is very small, normalised: $v = m/s = 0.015$.
- The design level is such, that out of all designs some 10% to 15% are considered to be inadequate.
- The quality of work execution is at a fairly high level and some 5% are judged to be inadequate.
- Outdoor environment under normal conditions results in the following modifications of service life:

**Table 3: Service lives and equivalent factors for different expositions**

| *exposition* | *East* | *North* | *West* | *South* |
|---|---|---|---|---|
| difference in service life | ± 0 years | - 2 to 3 years | -7 to 10 years | - 5 to 7 years |
| equivalent factor | 1.0 | 0.95 | 0.85 | 0.90 |

The equivalent factors have been estimated knowing that the expected service life lies somewhere between 50 and 60 years.

- In-use conditions do not have to be considered, as only direct mechanical destruction can be a result of use. These cases are however not of statistical significance.
- Maintenance level does not have to be considered, as basically no maintenance is required. Flat sheets are installed and do not receive any, or minimal maintenance only, throughout their service life.

### 6.2 Input data

For the calculation of service life, the following equation is set up according to the factorial method:

$$PSLDC = RSLC \cdot f_A \cdot f_B \cdot f_C \cdot f_E \qquad (2)$$

From above inputs, the following mean factors and standard deviations, or second moments respectively, are derived at:

- The density distribution of the factor for the quality of the component is, on the basis of the mechanical strength, set to a mean value of $f_A$ = 1.2. The standard deviation is, on the basis of the normalised standard deviation from production, set to $s_A$ = 0.02.
- The density distribution of the factor for the design level is set to a mean of $f_B$ = 1.1 with a standard deviation of $s_B$ = 0.12, resulting in some 13% of the cases being below 1.0, i.e. exhibiting insufficient quality.
- The density distribution of the factor for the work execution level is from experience asymmetric and a lognormal distribution is defined by a mean value (first moment) of $f_C$ = 1.1 and a second moment of $s_C$ = 0.06, resulting in some 5% of the cases being below 1.0, i.e. insufficient.
- The density distribution of the factors for the outdoor environment are set to the mean values $f_E$ in Tab. 3 above. The standard deviation is, for the sake of simplification, set to an estimated $s_E$ = 0.1 for all four expositions.

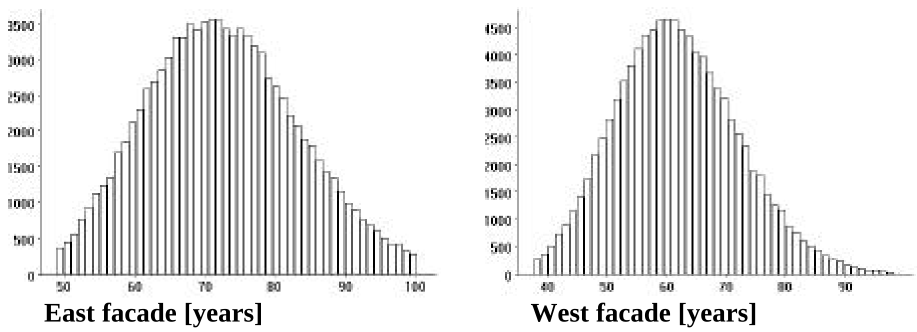
### 6.3 Calculation of service life

The calculation using VaP (1996) yields the following results for the predicted service life *PSLC*, basing on a *RSLC* of 50 years (deterministic value, for the purpose of this example):

**Table 4: Service lives and equivalent factors for different expositions**

| *Exposition* | *East* | *North* | *West* | *South* |
|---|---|---|---|---|
| *PSLC*, mean value [years] | 72. 6 | 69.0 | 61.7 | 65.3 |
| *m*, standard deviation [years] | 11.5 | 11.2 | 10.5 | 10.9 |
| for 16% damaged: *SLC* ≈ *PSLC* - *m* | 61 | 58 | 51 | 54 |

The distributions for the service lives of two facades are shown in Fig. 4.




*Note: Densities are the result of $10^5$ runs of a Monte Carlo simulation*

**Figure 4: Distributions of predicted service lives (*PSLDC*) for East and West facades**

Under the assumption, that damage to about one out of every six of the slates requires replacement of the entire respective cladding, the service life of the four facades (for a fractile of about 16% of damaged slates) is also shown in the above table, varying from 61 to 51 years.

### 6.4 Discussion of results

The differences in service life given from experience can be found in the results of the prediction. For the purpose of investment planning, the 16% fractile (or any other fractile deemed to be reasonable) seems to be a good indication of the point in time of replacement.

This example shows that even on the basis of relatively scarce input, quite sensible service life prediction are possible.

## 7 EXAMPLE 3: SIMPLIFICATION OF THE PROBABILISTIC METHOD

This example deals with chloride ingress into concrete. It is based on a paper using the probabilistic approach on the basis of error-functions (Edvardsen & Mohr 2000). In the paper, the authors compare the results of deterministic and of probabilistic calculations of service life of reinforced concrete structures in two climates (10°C and 30°C). The necessary concrete cover for a service life of 50 years has to be determined. A chloride content of 0.1 % chloride by mass of concrete defines the service life at the reinforcement.

The example demonstrates, that service life prediction using a relatively plain formula, different to the one in ISO 15686, 2000, can be done in the same way, again by introducing densities for the factors involved.

### 7.1 Procedure for simplified equation

In terms of an engineering design method, the following procedure is used:

1. The mean value of the chloride ingress depth is calculated as $x$ = 34 mm, using the equation for diffusion for a time of 50 years:

$$c(x,t) = c_s - (c_s - \cdot c_0)\left(1 - \operatorname{erf}\left[\frac{x}{2\sqrt{D\,t}}\right]\right) \qquad (3)$$

   where: $c$: concentration on chlorides, $c_S$: concentration at the outer face and $c_0$: initial concentration in the concrete. The front of the ingress is defined by the critical value $c = c_{crit}$ = 0.1 % of mass of concrete.

2. A simplified diffusion equation is set up for the depth $x$ of chloride ingress. By this, all constants in the equation of diffusion are rounded up into one single constant $K$:

$$x \approx K(c_s - c_{crit} - c_0)\sqrt{D}\,, \qquad (4)$$

3. The constant $K$ is calculated by solving the equation for the mean value $x$. Using the mean value of 34 mm this results in $K = 38{\cdot}10^3$ [$s^{0.5}$ / wt.-%].

**Table 5: Values used for the diffusion calculations**

| *variable* | | *distribution* | *mean value* | *standard deviation* |
|---|---|---|---|---|
| Surface chloride concentration | $c_s$ | Lognormal | 1.0 [wt-%] | 0.3 [wt-%] |
| Critical chloride content | $c_{crit}$ | Normal | 0.1 [wt-%] | 0.025 [wt-%] |
| Initial chloride content | $c_0$ | Normal | 0.01 [wt-%] | 0.002 [wt-%] |
| Eff. chloride diffusion coefficient (10°C) | $D_1$ | Normal | $1.0{\cdot}10^{-12}$ [$m^2/s$] | $0.1{\cdot}10^{-12}$ [$m^2/s$] |
| Eff. chloride diffusion coefficient (30°C) | $D_2$ | Normal | $4.0{\cdot}10^{-12}$ [$m^2/s$] | $0.4{\cdot}10^{-12}$ [$m^2/s$] |

### 7.2 Solving equation using distributions

The equation reads now as

$$x = 38\cdot 10^3 (c_s - c_{crit} - c_0)\sqrt{D} \quad \left[\sqrt{\text{s}}/\text{\% mass}\right] \qquad (4)$$

Solving this equation using the same density distributions as used in the detailed probabilistic solution (see table of Fig. 5), yields results as shown in Fig. 6. The slight skewness of the resultant density distribution is neglected for indicating standard deviations.

### 7.3 Fractile value required

The probability of exceeding the critical content $c_{crit}$ is set to 10% in Edvardsen & Mohr 2000 Assuming a normal distribution, the value of the fractile of 90% is derived at by adding λ = 1.28 standard deviations to the mean: $x_{90} = \bar{x} + l\,s$. These results are also shown in Fig. 6 and are compared to the exact values of the original paper. The mean values are identical as well as the standard deviation for Diffusion constant $D_1$. For Diffusion constant $D_2$, the fractile value of this prediction exceeds the exact value by some 5%.

This accuracy is deemed to satisfy the needs of the customer, bearing in mind, that all input values, although being set up as distributions, are still never perfectly exact.

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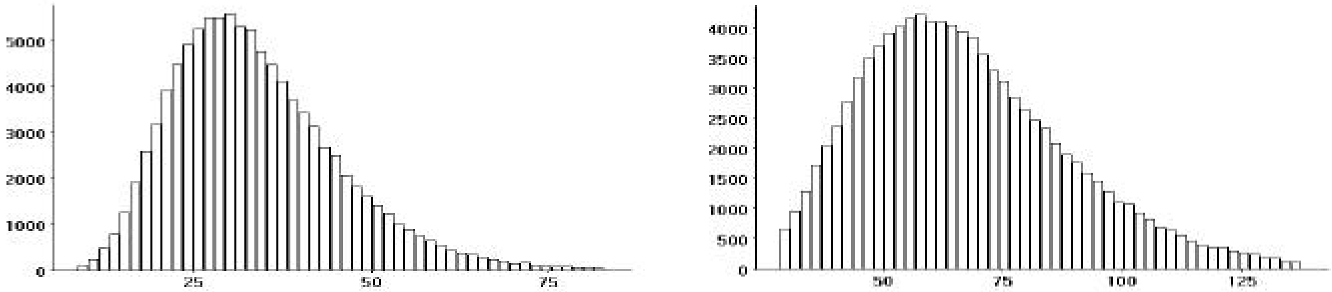
| diffusion constant | mean value | standard deviation | Fracile of 10% exceedance | |
|---|---|---|---|---|
| | | | eng. design method | Edvardsen & Mohr 2000 |
| $D_1$ | 34 mm | 12 mm | 49 mm | 49 mm |
| $D_2$ | 68 mm | 23 mm | 97 mm | 91 mm |




*x* [mm] for $D_1$ *x* [mm] for $D_2$

Note: Densities shown are the results of $10^5$ runs of a Monte Carlo simulation

**Figure 5: Engineering design method: density distributions for the depth of chloride ingress**

## 8 CONCLUSION

In order to calculate service life in the sense of the engineering design method EDM an equation containing the relevant factors at their relevant levels has first to be set up. Density distributions instead of plain factors in the equation for service life greatly improve the information content and the relevance of the results at a significantly reduced intellectual, mathematical and timewise input, compared to the quite often elaborate original equations.

By making the trail to the results clearly understandable, fewer errors will occur and fewer traps stepped into. Thus the method can be applied by the plain engineer yielding nearly as good results as finely tuned sophisticated probabilistic models.

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# Modifying Factors Of A Method For Estimating Service Life

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**Summary: This paper deepens the definition of agents that influence the service life of a building component and that converge in the determination of modifying factors of a mid-normal value. This value is at the base of a method, developed by Prof. Maurizio Nicolella of the D.IN.E., for the estimate of components service life in any environmental context.**

**The method is based on the possibility to assess the specific case as deviation from the mid value derived from field-collected data for assumed conditions.**

**The method leads to the redefinition of the factors and their relative weighted value for every considered building component.**

**The definition of these factors, the grouping in homogenous areas and their conversion in numerical values are developed and compared with the factorial method proposed in the code ISO 15686.**

**The method, suitable for every building components, is implemented with respected to the facade plaster and the definition of the numerical values for modifying factors will be carried out also basing also on programmed laboratory tests, which will be performed at D.IN.E. of the University of Naples.**

**From first results, namely from a first empirically chosen set of the numerical values, a software has been developed that allows to evaluate the most probable duration of the facade plaster in specific environmental, climatic and building configuration conditions.**

**Keywords. Service Life, Durability, Modifying factors, ISO 15686, Software for estimating service life.**

## 1. INTRODUCTION

Service durability and life of building materials or components is a research sector that has gained increasing attention over the last decade.

In this research context there is a study of a method for the estimate of components service life developed, in 1998, by Prof. Maurizio Nicolella of the D.IN.E. (Department of Building Engineering) of the University of Naples " Federico II ".

The method is based on the essumption that the service life of a building component can be estimated in any environmental context, considering the peculiarity of the case in object as shunting deviation from a mid-normal value obtained on an experimental basis.

## 2. THE METHOD[1]

Briefly, the method is based on the possibility to express the performances, relative to the specific conditions of the case-study, with reference to conditions assumed as medium case and as reference.

The formula that represents this condition analytically and that, therefore, is on the base of the method is the following:

[1] For a complete presentation of the method there is the article of Prof. M. Nicolella, published in the same proceeding of the congress: *Components Service Life: from field test to methodological hypothesis*

$$D_{pp} = D_{mn} \times \prod F_i$$

where:

**$D_{pp}$** is the value of the "most probable duration", that corresponds to the reliability of the considered building component in the assumed condition of use;

**$D_{mn}$** is the "mid-normal" value of the "duration", that is the reliability of considered building component in special conditions assumed as "mid-normal";

**Fi** are the modifying factors which, in a preliminary phase, are associated to every group of agents that influence the service life of the considered building component.

In particular, the **$D_{mn}$** value of a specific building component is the statistical mean of data taken from the study of some sample-buildings which are selected in accordance to specific criteria and associated from similar conditions assumed like "mid-normal" conditions.

The adopted criteria of choice are the following:

- quality and quantity of the available information;
- possibility to carry out monitoring activity;
- characteristic of previous operations;
- homogeneity of the influencing agents.

Whit regards to the assesment of modifying factors, it’s important to consider, for the evaluation of the duration statistically “most probable” of every building components, the agents that influence the reliability and order them in homogenous areas. To each of these homogeneous groups corresponds a modifying factors (Fi). The value of this factor is a function of the score (numerical value) that is attributed to the possible conditions of variation of the agents and function of the weight that is assumed from every agents group for the estimate of service life.

## 3. THE MODIFYING FACTORS

This paper focuses in depth on the problem of the factors’ determination.

The start is the choice and the organization in homogenous groups of the agents that influence the durability of a building component. Their choice is linked, firstly, to logic of the most relevant influence of an agent on the behavior of the considered component and so the agents determination, as their influence (weight) on the duration value, changes in relation to the component.

Secondly, the problems related to translating into numerical terms must also be considered, so that this conversion can be analytical or experimental and not only empirical.

Chosen the agents and grouped them in homogenous areas, we must analyze the conditions of variation of the generic agent (*i*), attributing them a score whose value results inversely proportional to the capacity to produce situations of reliability.

For any group of agents (1,2......,n) there is a distribution of scores relative to their variation conditions (Ci):

| Fattore 1 | |
|---|---|
| Condizione | Punteggio |
| $C_{i\,11}$ | $P_{i\,11}$ |
| $C_{i\,12}$ | $P_{i\,12}$ |
| ...... | ...... |
| $C_{i\,1mn}$ | $P_{i\,1mn}$ |
| ...... | ...... |
| $C_{i\,1j}$ | $P_{i\,1j}$ |

...

| Fattore n | |
|---|---|
| Condizione | Punteggio |
| $C_{i\,n1}$ | $P_{i\,n1}$ |
| $C_{i\,n2}$ | $P_{i\,n2}$ |
| ...... | ...... |
| $C_{i\,mmn}$ | $P_{i\,mmn}$ |
| ...... | ...... |
| $C_{i\,nk}$ | $P_{i\,nk}$ |

Every group of n factors has a range ΔF of possible values between the minimum and the maximum, that corresponds to **the weight** of any group in relation to the definition of the Dpp duration.

In every range there is the value mid-normal (that can not be in the center of the range). The modifying factor 1 corresponds to this value. The values comprised between this value and the minimum will assume smaller values than the unit (they produce a lessening of the reliability), those comprised between the same value and the maximum will have greater values than the unit (they correspond to better conditions for the reliability).

The total minimal score ($P_{min}$) is obtained from the sum of the minimums values of score of every table. The inferior limit of the range ($\Delta Fi$) of variation of the coefficient relative to the group corresponds to this score.

Analog ally, the score maximum total ($P_{max}$) derives from the sum of the maximum scores. The superior limit of $\Delta F$ corresponds to this $P_{max}$. The sum of the scores relative to the mid-normal conditions produces, instead, the value $P_{mn}$ , linked to modifying factor 1.

Defined:

D**Fi** the rejection that can assume the generic modifying factor (that correspondents to the relative group of influence agents);

$\mathbf{P_{max}}$ the total maximum value;

$\mathbf{P_{min}}$ the total minimal value;

$\mathbf{P_{mn}}$ the value of the condition medium-normal school;

the Fi value of the modifying factors, that corresponds to the *i* group of influence agents, will be comprised between the values $F_{min}$ and $F_{max}$ :

$$F_{min} = 1 - [DFi/DP \text{ x } (P_{mn} - P_{min})]$$

$$F_{max} = 1 + [\Delta Fi/\Delta P \text{ x } (P_{max} - P_{mn})]$$

Generically:

$$Fi = 1 + [\Delta Fi/\Delta P \text{ x } (P_i - P_{mn})]$$

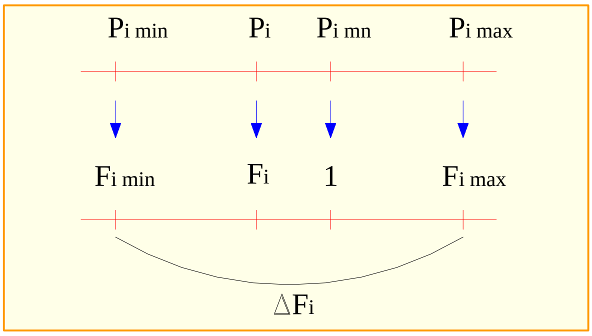


**Figure 1. Correspondence between scores and factors.**

The method is tested for various building components with an empirical attribution of the scores relative to the conditions of variation of the agents and the determination of the mid-normal value. In particular, as example for the plaster, the factors, organized in four groups, are the following:

| Climatic Agents | Environmental Agents | Configuration Agents | Technological Agents |
|---|---|---|---|
| Temperature | Exposure | Facade characteristic | Presence of protected elements |
| Daily Δt Temperature | Facing sea | | |
| Wind | Pollution | Shape | Critical points |
| Rain | Facing other buildings | Extension | Color |
| Snow | Vibrations | Lying | |
| Moisture | | | |

An interesting aspect regards the choice of the conditions of variations of several agents, whose thresholds of definition are chosen between significant values found in accredited studies or objective appraisals or, still, codified classifications, as an example, for running code.

For example, for the wind it adopts a classification proposed in an Italian code (D.M. 16 January 1996) dividing the national territory in 5 different zones. To any these 5 conditions it will be attributed a score whose entity is inversely proportional to the correspondent ability to induce situations of unreliability.

Therefore the main problem becomes the attribution of the numerical values (scores) to the various conditions of variation of the agents and the determination of the weight of every homogenous group.

The possible solutions are fundamentally following:

1. empirical attribution of the values based on the elaboration of data collected from case-study in which it's possible to point out similar behaviors.
2. empirical attribution of the values based on the analysis of data collected from studies and researches, indications and information from producers, constructors, technicians, etc., or other convenient sources.
3. experimental determination with the aid of laboratory tests that concur to estimate the relativity of the several conditions of variation of the influence agents regarding the case assumed as reference (values mid-normal), that is obtained always statistically by on-field collected data. With regard to this solution, taking advantage of the features of the equipment available near the laboratory of the D.IN.E., the working group of Prof. Nicolella is studying the possibility to develop a series of accelerated aging tests to estimate the behaviors of test pieces, opportunely manufactured, in variable conditions of climate and atmospheric pollution.

## 4. COMPARISON WITH THE FACTORIAL METHOD

The basic expression of the proposed method ($D_{pp} = D_{mn} \times PF_i$) recalls the formula expressed by the famous factorial method, introduced with the code ISO 15686 – part.1, which introduce, for the service life estimate (ESL), an equation defined by a reference value (RSL), modified from a series of multiplied factors that correspondent to the agents which can influence the service life of the considered component:

**ESLC = RSLC x Factor A x Factor B x Factor C x Factor D x Factor and x Factor F x Factor G**

where:

factor A: quality of components

factor B: design level

factor C: work execution level

factor D: indoor environment

factor E: outdoor environment

factor E: in-use conditions

factor G: rnaintenance level

The starting point of the factor method is the reference service life. It is a documented period in years that the component or assembly can be expected to last in a reference case under certain well-defined service conditions. be may be based on the following:

- data provided by a manufacturer, a test house or an assessment regime (for innovative components it will normally be based on the manufacturer's or supplier's exposure results); this may be a single figure or a distribution of typical performance;
- previous experience or observation of similar construction or materials or in similar conditions;
- Boards of Agreement in the EC state assessments of durability in their certificates or reports of national product evaluation services;
- some books which are available and which include typical service lives;
- building codes which may give typical service lives 1 of components.

As it's possible to note in both methods a reference value is modified by coefficients that translate the peculiarity of the specific case. But the genesis of this value, as the nature of the coefficients, is different.

Beiefly, the most important differences are:

- the nature of the value of reference (RSL and $D_{mn}$);
- the choice and the grouping of the influence agent ;
- the weight of every group of agents in relation to the value to estimate
- the genesis of the numerical value of the modifying factors.

In particular, for the first point, the mid-normal value is determined, statistically, by the mean of reliable collected on-field data in similar and objective conditions. This determination represents a sort of positive limitation to the relative freedom of choice that is expressed by the definition of the RSL. The ISO 15686 suggests to refer to all the possible sources available for the designer in order to support the adoption of a value that remains always empiricist.

A very interesting aspects, relative to the most evident differences, regards to the agents of influence and the relative modifying factors. Every factor corresponds to a group of agents whose combination changes in relation to the considered building component, and so the weight of the group of agents, for the estimate value, is linked with the nature of the same element. These approaches render the method in a position to adapting itself to different cases and so they concur to guarantee a greater reliability in the results.

Moreover, in the factorial method, the problem of the attribution of the values, that modify the reference service life, is directly linked to the factors and, in the greater part of the cases, it is expressed from the choice between a series of values limited and prefixed. In our case, instead, the value of the factors derives from the combination of the conditions defined for every agent of variation for the case in object and, therefore, it turns out specific for every different condition.

An other important consideration regards the responsibilities of the designer who uses the method as instrument for the service life estimate of a building component to the planning of opportune maintenance operations.

The factorial method foresees that the designer operates the necessary choices for the determination of the ESL. The present method assumes that the designer selects the conditions which are present in the case in consideration and, using the scores previously assigned, determines the value of the factor that translates the duration mid-normal into the value to estimate and to use subsequently.

Just in order to supply an easy instrument that renders friendly the use of the method, a software has been developed (later mentioned). This software interacts with the designer, simplifying the input operations, to guide him in the determination of the values of the most probable duration.

## 5. THE EXPERIMENTAL APPROACH

We have already expressed the plan to use the features from the instruments in equipment to the laboratory of the D.IN.E. for an experimental assesment of the scores to be assigned to the conditions of variation of the influence agents.

The tests will concur a partial resolution of the problem because only for some factors will it be possible to simulate the different conditions. The scope is not to determine the number of cycles to execute in order to verify the performance lowering beyond prefixed values and, therefore, an absolute value of duration, but to verify a relative data. Keeping constant the conditions of all the considerate agents, we study how the several conditions of variation influence the duration, to identify the corresponding scores. For example, considering the factor exposure, it would be possible to estimate how results influence a particular exposure, e.g. south, respect to another exposure, e.g. north, regard to the same limit of degradation, keeping other conditions constant.

This approach concurs to obtain an evaluation of the relative influences and therefore an experimental support to the choice of the scores, avoiding typical problems, linked to the test cycles, as the re-scaling. As a matter of fact, we don't determine an absolute value of duration but a <u>relative data</u> of intermediate type, that define the duration, also by a sound reference to the reality supplied from the mid-normal value, and so it is not necessary the re-scaling. In this way the traditional problems, linked to the aging accelerated tests, are exceeded, e.g. the report between the laboratory tests and the external reality.

A very delicate point is the choice of performance limits in correspondence of which arresting the test cycles.
The tests will have to be adequately calibrated and studied and constitute an object of a precise planning and programming in which the variables to be considered are multiple.

Briefly, the procedure to follow would have to be the following:

VALUATION OF THE FACTOR OF WHICH IS POSSIBLE TO DETERMINE THE VARIATION LAW
↓
REGULATION SEARCH ON THE EXECUTION OF THE TESTS
↓
STUDY OF THE AGENTS FOR THE DEFINITION OF THE TEST CYCLE
↓
ENVELOPE OF THE AGENTS
↓
COMPOSITION AND DEFINITION OF THE CYCLE:
(definition of objects to estimate the duration of the under-cycles; definition of the limits for every under-cycle; choice of the order for the execution of the under-cycles)
↓
STUDY AND COMPOSITION OF THE TEST PIECES
↓
EXECUTION OF PRELIMINARY TESTS FOR GAUGING OF THE CYCLE
↓
EXECUTION OF THE CYCLES
↓
LABORATORY TESTS ON THE TEST PIECES SUBORDINATES TO THE AGING CYCLES
(destructive tests – non-destructive tests)
↓
COLLECTION AND RECORDING DATA
↓
CONVERSION OF DATA IN USEFUL INFORMATION FOR THE METHOD APPLICATION

For the valuation of all the factors that cannot take advantage from results of the laboratory tests (v. technological factors), will continue to report to empirical attributions or to on-field collected data and reported to a meaningful sample of available examples (case-study).

The experimental approach, in the described terms, places the method in a particular position between the two proposals of ISO 15686.

The code, in fact, relative to the problem of the estimate of the service life, suggests two alternatives: the cited factorial method and the experimental approach that is characterized from aging tests of short and long term. The two methods are currently object of in-depth studies in sight of possible modifications. For this reason, three technical commission have been constituted to follow the research lines delineated from the two cited approaches and a third line that mediates the two solutions.

With the stochastic-experimental and the factorial methods, in fact, there is a new tradition represented from the engineering methods. The scope of the work of this task groups entices the following three steps:

Gain an overview on the main methods applied to research and large project using the scientific approach. These method often apply mathematical models and stochastic processing to the design data. In addition to the main task, look for modifications to the factorial method towards the scientific approach.

Define as to what an “engineering method” should be in terms of complexity of models applied and type and amount of data employed.

Set up engineering method preferably developed on and applied to typical case studies.

## 6. THE SOFTWARE

All the work carried out till now (experimental determination of the mid-normal values for some of the building components, like the external rended plaster, assigned with a score related to the variations in condition of the corresponding influencing agents, determination of relative modifying factors, etc.), flows towards the making of a calculation software of “the most probable duration” of a building component, in a given environmental conditions.

The software, through simple input, allows rapid data elaboration. The program used is VISUAL BASIC.

The designer must select the conditions relating to his particular case, relatively to all the influencing agents involved. The software selects the right score, that are then calculated to supply the duration value required. Once the value has been calculated it is possible to print up the results:

- Print of only the results
- Print of the windows of all the input factors
- Print of windows of the output related to the score factor.

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The software consents the interaction in *txt* form file in order to load other calculation parameters. Parameters relating to now types of plaster can also be loaded. This is possible by using the key “Open” that permits the loading of *txt* file containing the parameters of the new building components. By using “Save” it is possible to modify the value of various parameters and save a new setting with any name.

As example, some grids related to the software are below included.

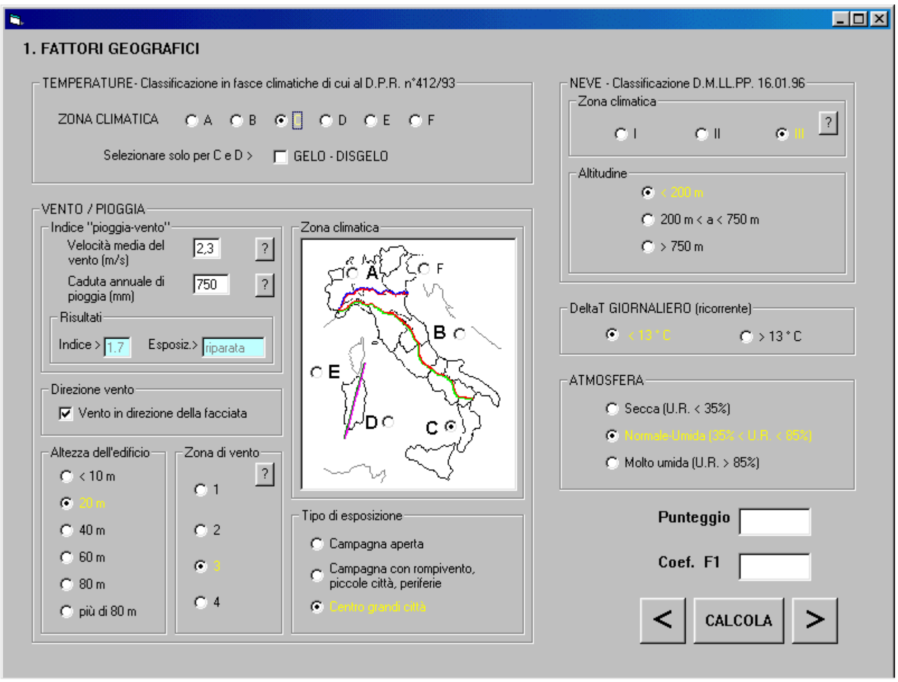


**Figure 2. Input form for climatic agents.**


**Figure 3. Input form for environmental agents.**

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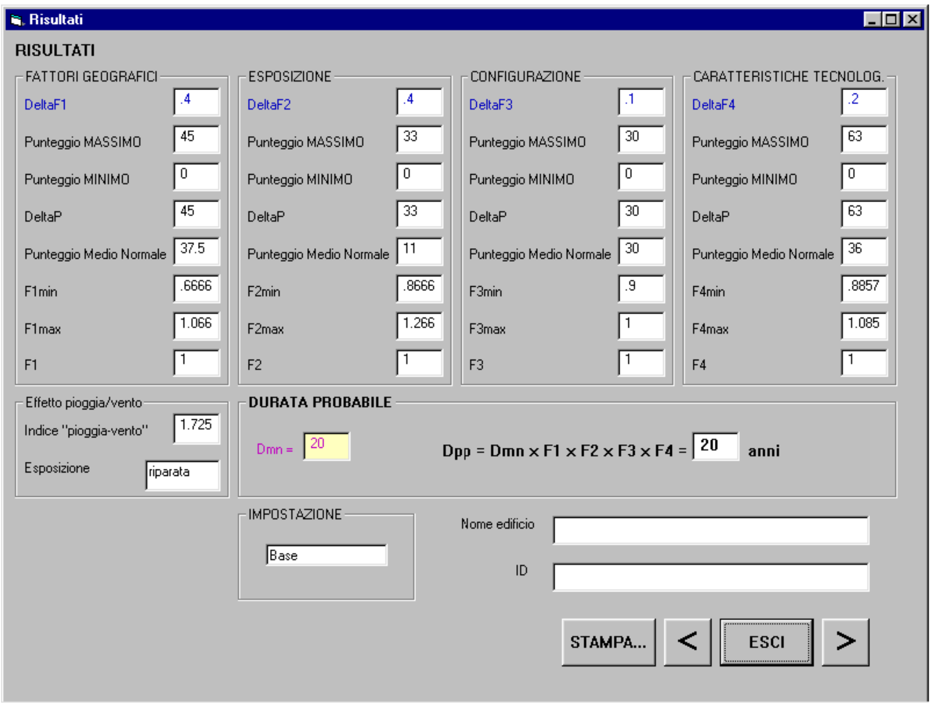


**Figure 4. Output form of results**

## 7. ACKNOWLEDGMENTS

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# Development Of Tools To Assist Performance-Based Appraisal Of Durability In Tropical Countries

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**Summary**: **A five-nation project has been addressing the issue of performance verification methods for metals in tropical climates. The project partners are CSIRO Australia, TISTR Thailand, ITDI The Philippines, CRC–IMS Vietnam and CRC Institute of Technology Indonesia. Over 20 sites were established across the five countries where climatic parameters and metal response parameters were determined. These sites could be consistently grouped on the basis of both climatic and material response parameters. This paper discusses the probable forms of sulfur compound deposition at the sites and then proposes a series of performance-based tests for each climatic grouping. The applicability of two of the performance-based tests is discussed. The development of a website to permit selection of test procedures is briefly outlined.**

**Keywords. Industrial pollution, atmospheric corrosion, tropical, performance test**

## 1 INTRODUCTION

This paper summarises results from a five-nation project including Australia, Indonesia, Philippines, Thailand and Vietnam. The project aims to develop performance verification methods for the durability of metallic components in South–East Asia. Performance verification methods can include outdoor exposures, chamber tests or analytical methods, however, this paper will concentrate on results from an exposure program and the use of those results to develop accelerated tests.

In order to characterise the environments and develop performance tests, it is necessary to define the critical environmental factors which affect the service degradation process so that these can be reproduced in the performance tests. Further, to validate any performance tests, it is necessary to demonstrate that the degradation mechanism in the performance test is the same as in the field. This was the aim of the exposure program. Given this knowledge, it was possible to group environments into a number of classes and characteristic environmental class types. A characteristic environmental class includes all those environments in which the same degradation mechanism occurs and the same environmental factors are critical. An information technology (IT) system has been devised whereby maps are used to define the geographic location of each environmental class in any location in the participating countries. Then, for each environmental class an appropriate performance-based test is proposed.

## 2 EXPOSURE PROGRAM

The position and characteristics of each site are given in Table 1. Experimental methods in exposing, measuring the environmental factors and analysing the plates are reported elsewhere (Cole *et al.* 1999). At each location, the following parameters have been measured: salinity by wet candle technique; $SO_x$ concentration by passive sample; time of wetness (TOW) by gold grid sensor fixed to a zinc plate; rainwater pH and conductivity; and total soluble particulate using air filters and $SO_4$ concentration by ion chromatography. Plates are exposed for every season and over a full year and, following the exposure, mass loss and mass gain are determined, as well as the oxide species using FTIR.

**Table 1. Australian and South–East Asian sites**

| *Country* | *Location* | *Type* | *Latitude/longitude* | *Salient features* |
|---|---|---|---|---|
| Australia | Low Island | | 16° 23'S, 145° 34'E | An island 15 km from coast |
| | Cowley Beach | Severe marine | 17°41'S, 146°06'E | On beach facing SE |
| | Cowley Beach-2 | Marine | 17°41'S, 146°06'E | 150 m from coast |
| | Moresby | Rural | 17° 38'S, 145° 01'E | 11 km from coast |
| | Butchers Creek | Highland | 17° 22'S, 145° 42'E | On very edge of tableland, 34 km from coast |
| | Walkamin | Highland | 17°08'S, 145°25'E | On tableland NE of Cowley Beach, 43 km from the coast |
| Philippines | Cabuyao | Industrial | 14°10'N, 121°07'E | Large industrial centre |
| | Bicutan | Urban | 14°30'N, 121°02'E | Metro Manila |
| | Baguio | Highland | 16°30'N, 120°33'E | 1500 m above sea level |
| Indonesia | Lembang | Highland (1100 m) | 6°81'S, 107°61'E | Near Bandung |
| | Gresik | Marine/heavy industrial | 7°16'S, 112°68'E | 0.5 km from coast near Surabaya |
| | Cikampek | Industrial | 6°41'S, 107°46'E | East Java, on coastal plain, near fertiliser plant |
| | Jebus | Marine | 1°66'S, 112°68'E | 10 m from high-tide mark |
| | Mentok | Marine/industrial | 7°16'S, 112°68'E | 20 m from coast (sheltered strait), adjacent to tin smelter |
| Thailand | Bangkok | Urban/industrial | 13°40'N, 100°30'E | On building near major road |
| | Rayong | Industrial | 12°41'N, 101°19'E | 500 m from sea |
| | Phrapradaeng | Industrial | 13°39'N, 100°34'E | Adjacent to river 10 km from sea |
| | Phuket | Marine | 7°50'N, 98°20'E | On seashore |
| Vietnam | Hanoi | Urban | 21°01'N, 105°52'E | 100 km from the sea |
| | Hue | Urban | 16°28'N, 107°36'E | 5 km from the sea |
| | Nha Trang | Marine | 12°20'N, 109°00'E | 10 m from the sea |
| | Ho Chi Minh City | Urban/industrial | 10°46'N, 106°43'E | 56 km from the sea |

## 2.1 Classification of environments

Burn *et al.* (1998) attempted to develop a system of general climatic zones supplemented by information on the critical environmental parameters controlling degradation. They define environments in terms of their humidity and temperature conditions. All sites would be classified as very humid (V) and Hot (H) or very hot (VH).

Burn *et al*. defined subclasses of external pollutant and UV exposure, viz.:

- Severe marine (SM) – airborne salinity exceeds a daily average of 300 mg/m².day.
- Marine (M) – average daily airborne salinity is between 60 and 300 mg/m².day.
- Severe industrial (SI) – airborne $SO_x$ level exceeds 200 mg/m².day.
- Industrial (I) – airborne $SO_x$ level is between 60 and 200 mg/m².day.
- Severe industrial and severe marine (SI+SM) – $SO_x$ level exceeds 200 mg/m².day and airborne salinity exceeds 300 mg/m².day.
- Marine or industrial influence (IN) – (a) airborne salinity is between 15 and 60 mg/m².day, or (b) airborne $SO_x$ is between 10 and 60 mg/m².day, or (c) rainwater pH <5.5.
- Benign (B) – all of the following must be met: (a) airborne salinity <15 mg/m².day; (b) airborne $SO_x$ <10 mg/m².day; and (c) rainwater pH >5.5.

While the classification of Burn *et al*. (1998) appears reasonable for our climatic data, the pollutant levels do appear a little high. Severe industrial greater than 50 $\mu g/m^3$, industrial 10–50 $\mu g/m^3$, and benign with salinity <15 $mg/m^2$.day and 10 $\mu g/m^3$ is more appropriate for our data. If such refined pollutant definitions are used, our sites can be defined as severe marine, marine, severe industrial, industrial and benign, and would be labelled VH-SM, VH-M, VH-SI, VH-I and VH-B respectively. Some locations, notably Rayong, vary with season (from VH-I to VH-M).

### 2.2 Exposure program results

Full experimental results are presented elsewhere. Table 2 presents the range of conditions for each exposure category, and Table 3 presents the range in surface response to these conditions. The probable particulate composition in Table 2 is estimated by ratios of elements determined from ion chromatography (IC) analysis of collected particulate. The n value for zinc refers to the value of time dependence derived by a comparison of the seasonal to annual corrosion rates according to the following formula:

$$ML(t) = Ml_o * t^n$$

where ML(t) is the mass loss in t years, and $Ml_o$ is the mass loss over one year.

The grouping does appear to partition the climate and some of the metal response parameters in a consistent way. Thus, the severe marine sites show high TOW and significant rainwater conductivity, but relatively high pH values; while the severe industrial sites exhibit high gaseous $SO_x$, relatively high particulate levels (measured by total soluble particulate (TSP)) with high sulfate level, and a particulate composition consistent with $(NH_4)HSO_4$. Interestingly the SI sites show more variable TOW values than the SM sites. In general, the metal response fits into the same patterns as the climatic parameter, although it is notable that the zinc corrosion rate measured annually or seasonally varies only slightly for the non-marine sites. The variations in the surface chemistry of the oxide correspond to variations in environmental conditions, with zinc hydroxysulfate and carbonates occurring in severe industrial and industrial sites, whilst zinc hydroxy chlorides are also apparent at marine sites. In the benign sites only zinc hydroxy carbonate is evident.

**Table 2. Climatic characteristics for different climatic subclasses**

| *Subclass* | *Location* | *Salinity* (mg/m$^2$.day) | *SO$_x$* (mg/m³) | *TOW* (%) | *Rainwater pH* | *Rainwater conductivity* (mS/cm) | *TSP* | *SO$_4$ conc. in particulate* | *Probable particulate* |
|---|---|---|---|---|---|---|---|---|---|
| VH-SM | Cowley Beach, Low Island, Phuket (1 season) | >100 | <10 | 90–100 | 5–6.5 | 60–220 | 20 | 1.5 | MgCl, NaCl |
| VH-M | Cowley Beach-2, Phuket, Jebus | 30–100 | <10 | 60–100 | 5–7 | 20–40 | — | — | NaCl |
| VH-SI | Prap, Gresik | <30 | >50 | 30–90 | 5–7 | 30–100 | 65 | 7 | $(NH_4)HSO_4$ |
| VH-I | Rayong, Bangkok, Bicutan, Cabuyao, Ho Chi Minh City | <30 | 10-50 | 20–80 | 4–5 Phil.<br>5–7 Thai. | 30–40 Phil.<br>30–70 Thai. | 30–50 | 2–5 | $(NH4)_2SO_4$, $(NH_4)_3H(SO_4)_2$<br>$(NH_4)HSO_4$ |
| VH-B | Bagiou, Hanoi, Walkamin, Lembang | <30 | <10 | 60–100 | 4–5 | 10–20 | 20 | 2 | $(NH_4)_2SO_4$ |

**Table 3. Response characteristics for different climatic subclasses**

| *Subclass* | *Location* | *Annual steel corr-osion rate* (mm/year) | *Annual zinc corro-sion rate* (mm/year) | *Seasonal steel corro-sion rate* (mm/year) | *Seasonal zinc corro-sion rate* (mm/year) | *N values for zinc* | *Zinc mass gain/ loss ratio* | *Corrosion product* | *Corrosion product morphology* |
|---|---|---|---|---|---|---|---|---|---|
| VH-SM | Cowley Beach, Low Island, Phuket (1 season) | 280 | 8 | 70–400 | 3–30 | 0.2 | 0.3–0.5 | Hydroxysulfate/ chloride/carbonate | Dry gel/ crystalline |
| VH-M | Cowley Beach-2, Phuket, Jebus | 30–40 | 1.5–2 | 40–70 | 2–6 | 0.4–0.5 | 0.3–0.5 | Hydroxysulfate/ chloride/carbonate | Dry gel/ crystalline |
| VH-SI | Prap, Gresik | 30–40 | 1.5–3 | 50–100 | 1.5–5.0 | 0.7–0.8 | 0 | Hydroxysulfate/carbonate | Crystalline |
| VH-I | Rayong, Bangkok, Bicutan, Cabuyao, Ho Chi Minh City | 20–40 | 1–2.5 | 50–90 | 1–3 | 0.5–0.8 | –0.1 to 0.1 | Hydroxysulfate/carbonate | Crystalline |
| VH-B | Bagiou, Hanoi, Walkamin, Lembang | 5–15 | 0.4–2 | 6–40 | 1–3 | 0.2–0.6 | 1–3 | Hydroxy carbonate | Dry gel |

### 2.3 Relative importance of different deposition mechanisms

In order to develop appropriate models and test procedures to duplicate atmospheric corrosion, it is necessary to estimate the relative role of the different forms of sulfur compound deposition onto plate surfaces. The relevant deposition mechanisms are gaseous deposition into moisture films, particulate deposition and deposition in rainwater. Firstly consider the deposition rate of particulates, given an exterior concentration of ammonia sulfate particles of 20 μg/m³ (dry weight), which is towards the upper value of the particulate concentrations given in Table 2. Analysis by Cole et al. (2000) indicates that given a plate exposed at an angle of 45°, deposition onto the vertical plane will dominant and be defined by the following relation (Sehmel 1980):

$$F = -\eta V_w C$$

where $F$ is the dry deposition flux, η is the collection efficiency, and $C$ is the local concentration of depositing species.

Collection efficiency approaches 1 for large particles and 0 for small aerosol particles. The exact size distribution of the particulates at the sites detailed in this report has not been quantified, however, typically (Wall 1988) the size distribution of ammonia and sulfate aerosol will have two peaks around 0.7 and 3 μm, with the smaller particles having three times the concentration (by volume) of the coarse particles. Sehmel (1980) indicates that typical efficiencies for 0.7 and 3 μm particles would be approximately $10^{-4}$ and $10^{-3}$ $ms^{-1}$.

If we then assume that the mass of fine particles is 15 μg/m³ and that of the coarse particles is 5 μg/m³, then the flux at a typical wind speed of 3 m/s will be 0.12 and 2.7 $\mu g/m^2 s^{-1}$ respectively for the fine and coarse particles. This then gives a deposition rate per hour of 10,150 μg/m² or 108 μg/dry weight per hour onto a plate of size 100 × 150 mm at an angle of 45° to the vertical.

If this amount of particulate were to deposit into a moisture film, it would then change the composition of that film to an extent that depends on the particulate composition and thickness of the moisture film. In Table 4, the change in $[H^+]$ and $[SO_4^{2+}]$ in M/L per hour is given.

No direct measurements have been made on the composition of moisture films on plates, however rainwater compositions give pH values from 4.2 to 6.5 and $[SO_4^{2+}]$ from $3 \times 10^{-5}$ M/L to $7.6 \times 10^{-4}$ M/L. Cole *et al*. (2001) estimates that the concentration in a moisture layer formed where the atmosphere is $CO_2$ at 400 ppm, $SO_2$ at 75 ppb, $NH_3$ at 20 ppb, $O_3$ at 200 ppb and $H_2O_2$ at 10 ppb, would have $[H^+]$ of $1.7 \times 10^{-6}$ M and $[SO_4^{2+}]$ of $1.2 \times 10^{-4}$ M or $0.63 \times 10^{-4}$ M of [H] and $0.34 \times 10^{-4}$ M of $[SO_4]$ without the presence of the $NH_3$. Thus, the deposition of particulate onto a moisture film will substantially increase the ionic content and acidity of the film. If the moisture film persists for a number of hours, then particulate deposition is likely to dominate over gaseous absorption.

**Table 4. Effect of particulates on moisture film composition**

| *Particulate concentration (mg/m³ dry weight)* | *Particulate type* | *Moisture film thickness (mm)* | *$[H^+]$ M/L* | *$[SO_4^{2+}]$ M/L* |
|---|---|---|---|---|
| 20 | $(NH_4)_2SO_4$ | 100 | — | $7.5 \times 10^{-4}$ |
| 20 | $(NH_4)_3H(SO_4)_2$ | 100 | $4 \times 10^{-4}$ | $8 \times 10^{-4}$ |
| 20 | $NH_4HSO_4$ | 100 | $8.5 \times 10^{-4}$ | $8.5 \times 10^{-4}$ |
| 20 | $H_2SO_4$ | 100 | $20 \times 10^{-4}$ | $10 \times 10^{-4}$ |
| 20 | $(NH_4)_2SO_4$ | 500 | — | $1.5 \times 10^{-4}$ |
| 20 | $(NH_4)_3H(SO_4)_2$ | 500 | $0.8 \times 10^{-4}$ | $1.6 \times 10^{-4}$ |
| 20 | $NH_4HSO_4$ | 500 | $1.7 \times 10^{-4}$ | $1.7 \times 10^{-4}$ |
| 20 | $H_2SO_4$ | 500 | $4.0 \times 10^{-4}$ | $2.0 \times 10^{-4}$ |
| 5 | $H_2SO_4$ | 500 | $1 \times 10^{-4}$ | $0.4 \times 10^{-4}$ |

### 2.4 Comparison with rainwater

It is also possible to compare the relative extent of acid deposition by rainwater as compared with particulate deposition. This has been undertaken and is presented in Fig. 1, where curves are derived assuming that the particulate consists wholly of $(NH_4)_3H(SO_4)_2$ or $H_2SO_4$ and that the rainfall per day is either 5 or 50 mL. The lower rainfall is around the average rainfall per day across the sites, whilst the higher is typical of the monsoonal period. On the same graph, the average conditions at a number of the exposure sites are listed. It is evident that out of the monsoonal period particulate deposition is likely to be

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dominant. During the monsoonal period for some sites with rainwater pH around 4 and moderate particulate levels, rain deposition will be the more important mode of acid deposition.

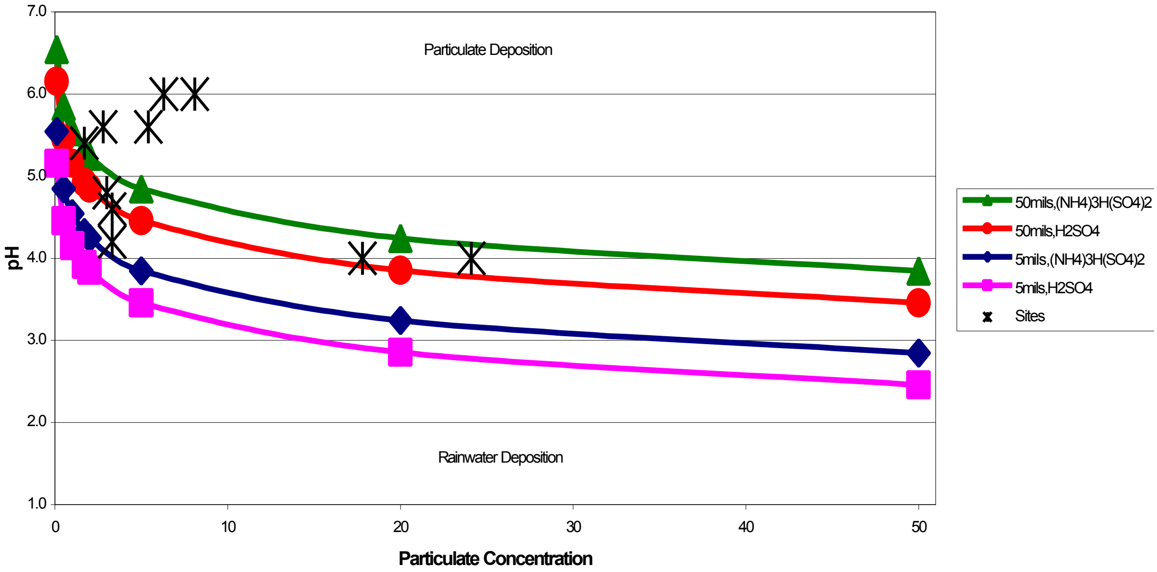


**Figure 1. Dominant deposition mechanism**

## 3 PERFORMANCE TESTS

The previous analysis indicates that particulate and rainwater deposition are likely to be the main forms of deposition of sulfur compounds. Given this performance, tests based on wet deposition have been devised. If these performance tests are to be an acceptable means of estimating service life then:

- The degradation mechanism in short-term tests and service conditions are the same.
- A consistent acceleration factor between short-term tests and service conditions being simulated exists and can be well defined.
- The short-term tests must incorporate the worst conditions under the environment that is being simulated.

In particular South–East Asia would be both safe and cheap. Tests to meet the variety of environments existing in South–East Asia have been developed and are summarised below.

### 3.1 Test 1 – acidified rain test

Condition simulated: tropical, industrial (VH-SI, VH-I)

Test design

Rainwaters of defined chemistry are sprayed onto components or metal samples for defined lengths of time per day. In non-rain periods a constant (high) RH is applied. The amount of pollution deposited via wet deposition is set to reflect the amount of pollution that would be deposited in the service environment under investigation. A constant humidity is justified by the high humidities found, particularly in the wet season in tropical countries.

Range of applicability

The rainwater composition can be varied to cover different levels of anthropogenic pollution; the constant RH can be varied somewhat (from 80–100%). This test is probably most applicable to those components which are not exposed directly to the sun.

### 3.2 Test 2 – acidified rain test with cyclic humidity

Condition simulated: tropical (subtropical), industrial (VH-SI, VH-I, HH-SI, HH-I)

Test design

The test design is the same as test 1 except that after a given number of rain and high humidity cycles, a relatively low RH cycle is included. This test has the added feature of a drying cycle which aims to simulate the drying of components directly exposed to the sun.

Range of applicability

The test would be applicable to the tropical/industrial conditions of test 1, but the additional drying cycle indicates that it will be applicable to sun-exposed conditions, and further may be applicable to environments such as subtropical environments where conditions are not uniformly humid. The value of RH in the drying cycle could be adjusted to reflect the degree and strength of exposure to the sun and/or the extent of variations in ambient RH.

### 3.3 Test 3 – airborne salinity/acidified rain test with cyclic humidity

Condition simulated: tropical, subtropical, industrial, severe marine, marine etc. (VH-SI, VH-I, VH-SM, VH-SM+SI, VH-I, HH-SI, HH-I, HH-SM, HH-SM+SI, HH-I)

Test design

The test permits deposition of salt aerosol, high humidity, drying cycles and acidified rain. The exact sequence and magnitude of any of the components can be modified to reflect the environment under consideration.

Basic principle

All the environmental agents are applied in a manner that reflects the actual service conditions. Thus, the dosage of salt aerosol given would reflect the range of airborne salinity commonly found (5–300 mg/m².day) in moderate to severe marine environments. It may be possible, however, to increase the frequency of incidence of saline deposition (probably double) without altering the corrosion mechanism. The chemistry of the acidified rainwater will, in non-industrial environments, simply reflect the chemistry of the rainwater of the service environment under study. However, in industrial environments wet deposition can be used as a surrogate for other forms of deposition.

Range of applicability

Tests would be applicable to both sun-exposed and sheltered components in tropical and subtropical environments.

### 3.4 Test 4 – airborne salinity test with high humidity

Condition simulated: tropical, severe marine, marine, marine influence (VH-SM, VH-M, VH-I)

Test design

The test permits the controlled deposition of airborne salinity in a high RH environment.

Range of applicability

Limited to tropical/marine and non-sun-exposed conditions.

### 3.5 Full details of Test 1 – acidified rain test

Five tests were carried out over a period of six weeks, with a daily application of rainwater (7 hours a day) followed by constant 90% RH and 30°C for the remainder of the day. Rainwater pH and ionic concentration were varied with the conditions in each rain test being given in Table 5. In test A, de-ionised rainwater was dosed at the following levels: $SO_4$ 0.054 M; $NO_3$ 0.019 M; Cl $3 \times 10^{-5}$ M; K 0.19 M; Mg 0.054 M; Na $3 \times 10^{-5}$ M; while pH was controlled by the addition of acetic acid. The flow rate onto the plates was estimated to be 1.2 L/min.m². This is equivalent to a deposition rate of 5.2 g/m² of $SO_4$ and 1.2 g/m² of $NO_3$ per day. The mass loss of the exposed plates is given in Table 6, whilst the mass gain (prior to etching) for the zinc and galvanised steel is given in Table 7. In addition, in Table 7 the ratio of mass gain to mass loss is given.

**Table 5. Conditions for rain tests**

| *Test* | *Salt concentration relative to test A* | *pH* | *Conductivity* |
|---|---|---|---|
| A | × 1 | 6.9 | 58 |
| B | × 1 | 4.6 | 52 |
| C | × 1 | 5.6 | 53 |
| D | × 5 | 4.6 | 170 |
| E | × 10 | 4.6 | 222 |

**Table 6. Mass loss (μm/year) after rain tests**

| Test | *Mild steel* | *Cu-containing mild steel* | *Galvanised steel* | *Zinc* |
|---|---|---|---|---|
| 1 | 61 | 65 | 0.7 | 2 |
| 2-a | 140 | 151 | 16 | 20 |
| 2-b | 146 | 146 | 26 | 21 |
| 3-a | 411 | 356 | 10 | 46 |
| 3-b | 423 | 351 | 15 | 44 |

**Table 7. Mass gain (μm) after rain tests**

| *Test* | *Galvanised steel* | *Zinc* | *Mass gain/mass loss for galvanised steel* | *Mass gain/mass loss for zinc* |
|---|---|---|---|---|
| 2-a | 0.63 | 0.59 | 0.34 | 0.25 |
| 2-b | 0.19 | 0.58 | 0.06 | 0.23 |
| 3-a | 0.01 | 1.3 | 0.01 | 0.24 |
| 3-b | –0.12 | 1.4 | –0.07 | 0.27 |

Surface analysis was undertaken on galvanised steel and zinc exposed in test A. In the case of the galvanised steel, the surface was eroded with isolated islands of hexagonal crystals (as indicated by SEM). In the case of zinc, the corrosion product was dry gel. IR spectra contained absorptions associated with sulfate and hydroxide which is consistent with the presence of zinc hydroxy sulfate. Further, the spectra of the zinc in the rain room is very similar to that of Bicutan and other polluted sites in South–East Asia (refer to Table 6).

### 3.6 Details of Test 3 – airborne salinity/acidified rain test with cyclic humidity

This test consists of repetition of a cycle with the following components:

a) Salt spray of 625 mg/m² (duration approximately 30 minutes).

b) High humidity for 32 hours.

c) Dry humidity of 55% for 12 hours.

d) Repeat (a) to (c) four times.

e) Acidified rain for 7 hours (pH = 4.5).

f) Hold at 80% RH until total cycle time is 168 hours (7 days).

g) Repeat cycle.

This test is aimed at duplicating conditions at industrial and marine sites (such as Rayong). In Table 8 the mass loss at Rayong is compared with that of six cycles of Test 3, and then an estimate of the equivalence of a number of cycles in the tests to a year in service at Rayong is made. It is found that for zinc a medium value is five cycles which is equivalent to one year in service. The surface chemistry at Rayong was zinc hydroxy carbonate with some zinc hydroxysulfate and zinc hydroxy chloride, whilst that of the rain test was a mixture of zinc hydroxy carbonate and zinc-acetate. Acetic acid was used initially to obtain the required pH. Tests are now underway to assess if replacement of acetic acid with sulfuric acid in the rain of Test 3 will promote the formation of a zinc sulfate.

**Table 8. Summary of corrosion data**

| *Materials* | *Exposure* | *Mass loss (mm/cycle)* | *Cycles per year* |
|---|---|---|---|
| Mild steel | Test 3, 6 cycles | 2.9 | 14–16 |
| Zinc | Test 3, 6 cycles | 1.0 (μm/year) | 1–9 |
| Mild steel | Rayong | 42–46 | |
| Zinc | Rayong | 1–8.4 | |

### 3.7 Discussion of rain room test

The rain room tests generate very similar corrosion products to the field exposures in tropical industrial zones. This is indicated by the similar FTIR spectra which indicate that both develop zinc hydroxysulfate products, have similar morphologies and both have light corrosion products which give strong indications of oxide stripping. Further evidence of stripping are the low ratios of mass gain to mass loss apparent in both the rain and the field tests.

The analysis of the sulfur deposition onto plates provides a rationale for the use of a rain test as a performance evaluation test. We have argued that sulfur deposition in the field primarily occurs as aerosol sulfate, and that corrosion occurs when this aerosol wets. The wetting of a sulfate aerosol will produce a moisture layer similar to that produced by the wet deposition of sulfate in the rain test. In the service environment, a number of processes occur, including aerosol deposition and moisture layer formation, surface drying and rain washing. In the current rain test, effective aerosol deposition/moisture layer formation and rain washing are combined, while significant dry periods are not included. Both the combination of sulfate deposition with rain washing and the lack of a drying cycle will restrict the development of corrosion films. This will accelerate the corrosion in the rain test relative to the service environment, but does not appear to alter the mechanism of corrosion.

The corrosion rates in the field tests and in rain Test 1-A for zinc are comparable, however, the corrosion rates in rain Tests 1-B and 1-C are respectively 10 to 20 times higher than the field rates. Thus, exposure in the rain tests for 6 weeks appears equivalent to at least 60 or 120 weeks in the field. Further analysis is required to determine the factors promoting the acceleration in the rain tests, and more detailed spectrographic work is needed to fully verify that the mechanisms of corrosion in the rain tests are similar to the field. Nevertheless, initial work indicates that the rain tests provide an acceptable performance test for metal coatings in tropical industrial areas.

## 4 IT SYSTEM

Having characterised and grouped the environment into classes and devised performance tests for each class, it is then necessary to provide a tool to enable the appropriate classification and test selection for any location. A website has been developed to do this. It is based on hazard maps developed where the user selects the location of interest, and the system then defines the climatic classification and suggests appropriate tests.

## 5 CONCLUSIONS

This paper summarises data from the five-nation research project into atmospheric corrosion in South–East Asia. On the basis of spectrographic and morphological evidence, the division of sites into three groups is proposed, viz. inland, marine and industrial sites. This grouping is also consistent with other surface response parameters and is found to reflect the type of pollutant at a site. Rain tests were developed to simulate exposure in industrial sites and a salt rain test was developed to simulate exposure in industrial/marine sites. The rain test appears to develop surfaces that have the same characteristics and corrosion products as the industrial sites.

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# Components Service Life: From Field Test To Methodological Hypothesis

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**Summary**: **In order to constitute the implementation base for a calculation model of the external plaster Service Life, data regarding 100 sample-buildings are collected through extensive field test.**

**The model represents a sample application of a general methodology, that estimate the component service life as a deviation from the standard value, modifying the factors referred to the influencing agents to be evaluated in advance for each building component.**

**The collected value, statistically elaborated, show low mid value dispersion, thus demonstrating the reliability of so-called "mid-normal" value. A software for managing the modification factors and quickly calculating the external plaster service life has been developed.**

**More recently, experimental tests have been started in order to relatively (not absolutely) assess the modifying factors, that at present are usually evaluated only through field tests. The results are assumed to be usable from middle year 2002.**

**Keywords. Service Life, Durability, Modifying factors, Influence agents.**

## 1 INTRODUCTION - PRINCIPAL INSPIRATION OF THE METHOD

The method, raised from the results of a study which has been carried out during a long period of time (approximately 15 years, estimated to be sufficient to provide for a field-collected data), aims at offering, to people interested in programmed maintenance in the building sector, a suitable instrument to estimate the durability of the building components: in other words, a methodological criterion in order to identify the expiration terms to be inserted in *the planning* of the maintenance plan, which correspond to appropriate intervention.

To achieve this fixed aim, it seems suitable to refer to the "*in service*" behaviour of the building components, because a substantial data base could be available, and also to avoid all problems concerning data from laboratory tests or theoretical modelling.

Before the experimental assessment, a theoretical analysis has been carried out, basing on the results obtained by the international scientific community, that has allowed not only the implementation of the model but also some choices of a quantitative nature regarding the "scores" to be assigned in the first instance to the influencing agents taken into consideration.
All this considered possible to define "semi-prob" this methodology which compares the probilistic theory on the reliability and the objective data of the field tests, trying to obtain results which have the requirement to be available for the operators and of general validity, (whose lack in the past has often prevented from exporting valuable results - but obtained in restricted fields - also in other contexts, or at least it has left wide doubts on such possibility.

For what concerns the use of the laboratory data, a few observations can be done:

- the hypotheses formulated on the use of the results of the buildings sampling, executed for obtaining a database, seem to be confirmed from their insufficient dispersion. As a matter of fact, in general, the results - converging around to the assumed mid -value - seem to attest the reliability of a criterion aimed, at characterizing "mid-normal " situation;
- caution has been adopted in the use of the laboratory data, since they refer a few building components in context conditions, for which an uniformity of exercise could be supposed (not neglecting the same criterion for the connotations of technological and typological character of the buildings), and permitting ulterior appraisals in case of data dispersion;
- on the basis of observations made by some research workers, in the proposed model $F_i$ factors, which trigger unrealiability of the experimental data, have been considered (besides of pathological states on the constructive

element). In fact, the experimental data often accumulate and statistically elaborate information related to non homogeneous situations, mainly due to the presence of some of $F_i$ factors , not accounted for.

In the proposed methodology, have been overcome problems regarding the relationship between aging laboratory tests and in field tests, just through the consideration of the "influencing factors". These are the most influential agents on the durability of the building components, as "generators“ of modifying factors of an experimental data, collected in almost constant conditions concerning these agents, and then - in an equally decisive way - using data which haven’t been derived from an experiment "*with techniques of natural exposure*" but considering the effective "in service” behavior.

The carried out methodology tries to consider two various requests:

- The use of the field-collected data, thanks to a sampling executed in the course of the years on buildings, materials and interventions;
- the possibility to generalize and therefore to export results otherwise referred for specific fields, through a criterion declared and aimed to adapt the model to the various contexts.

We assume is that the durability of building components can be estimated starting from base value, defined as "mid - normal", to which a performance level corresponds, corrected by weight factors. These factors measure the deviation between the assumed value and the case in examination, also on the basis of the criteria followed by other research workers.

It is said to have:

$$\boxed{D_{pp} = D_{mn} \times \prod_{i}^{n} F_i}$$

where:

**$D_{pp}$** the value of the "*most probable duration*", correspondent to the durability of the considered building components, in the assumed conditions (period in which the performances are above a predetermined value, because the probability it can assume lower values is less);

**$D_{mn}$** the value "*mid-normal*" of the "*duration*", that is the durability of the considered building components in the conditions assumed as "*mid-normal*"

**F** n the modifying factors of $D_{mn}$ correspondents to many influencing agents or groups of influencing agents.

During the work phase, that such methodology requires, should be reliably defined:

a) the performance level under which it is necessary an intervention, or - better - various decreasing performance levels to which different interventions of increasing intensity correspond, being in the building field non bi-stable components;

b) the value "mid-normal" of the non reliability;

c) the agents which influence the reliability, in other words the causes that can preside to the appearance of anomalies in respect to the physiological obsolescence of the building component;

d) the criterion of grouping of the mentioned agents;

e) the modifying factor to associate to the agents, each of a deliberate function of assigned weight, that is incidently shown in reality;

f) the criterion for the selection of the study cases;

g) the modalities of relief of the informations which have collected for the prechosen study cases;

h) the study cases correlated to $D_{mn}$ in other words those thought in "mid-normal" situation;

i) the criterion of decomposition of the building organism in parts, refer to research.

j) the building components to adopt as a meaningful sample of starting, thinking opportune to begin from a sub - ensemble of the entire building organism;

k) the study cases that allow the proportion and the calibre of the factors $F_i$ in a successive phase, during which it is necessary to determine the values of the single modifying factors, through the "earlier" reading of ulterior cases to select in different conditions with respect the "mid-normal"ones.

## 2 CHOICE OF THE BUILDING COMPONENTS OBJECT OF THE RESEARCH

It seemed suitable to start the study and to illustrate the proposed methodology on a limited sub - ensemble of building components, therefore to be able to easily finalize the research and to test the results.

To this aim the elements which constitute the external covering of the building have been selected, that is to say – according the UNI 8290 classification - elements pertaining to the following classes:

| | |
|---|---|
| *TECHNOLOGICAL UNIT CLASSES* | 2. CLOSURE |
| *TECHNOLOGICAL UNITS* | 2.1 VERTICAL CLOSURE |
| *TECHNICAL CLASSES OF ELEMENTS* | 2.1.1 vertical outside walls<br>2.1.1.1 Support<br>2.1.1.2 covering<br>2.1.2 external frame |

| | |
|---|---|
| *TECHNOLOGICAL UNIT CLASSES* | 2. CLOSURE |
| *TECHNOLOGICAL UNITS* | 2.4 SUPERIOR CLOSURE |
| *TECHNICAL CLASSES OF ELEMENTS* | 2.4.1 Roofings<br>2.4.1.1 flat continuous roofing |

In particular, on the basis of the ulterior decompositions which is possible to define in the inside of the technical elements classes, the following sub-classes of technical elements have been considered:

*2.1.1.2 Coverings of facade*

2.1.1.2.1 with natural stone / marble

2.1.1.2.2 with bonded materials (glass like, ceramic, etc.)

2.1.1.2.3 with external rended plaster / coloured

*2.1.2.1 external frames in wood*

*2.1.2.2 external frames in iron*

*2.1.2.3 external frames in aluminum*

*2.1.2.4 external frames in PVC*

*2.4.1.1 flat continuous coverings*

2.4.1.1.1 Practicable

2.4.1.1.1.1 covered with ceramic materials

2.4.1.1.2 Non practicable

2.4.1.1.2.1 covered with bituminous materials

The choice of the building components listed above derives from the following reasons:

- they are elements for which both the sampling and the observation of the process is easier;
- the life cycle in a generalized manner is not too long, and in any case allows to read and to interpret the results in reasonable time, at least regarding the fixed period of experimentation;
- previous research data concerning old sampling could be used;
- these are elements for which the influencing agents, tough numerous, have not been affected by other variables of a problematic valuation, and therefore they are very difficult to govern to which the ones used are suitable mainly to statistical elaboration.

Subsequently, it has been considered opportune to deepen the illustration on a single constructive element (external plaster), creating a software of management of necessary data and of duration calculation.

## 3 DEFINITION OF THE PERFORMANCE LEVELS

For each one of the 3 categories of building components the recurrent anomalies which affect the service life and the damage type, have been characterized.

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It is best underlined and considered, in the present study, the anomalies consequent to errors made during the phase of planning and/or realization, referring to what happens in the course of a normal process of aging due to natural phenomena.

In order to identify the correspondent performance levels and to program the correspondents intervention: because, as exposed in the present chapter, it is pointless to consider the life cycle in absolute in the case of a maintenance program, being the mainly building components mostly bi-stable, it is important to report such concept also to intermediate stages as expression of lower performances then the initial, yet. In order to determine parameters such as reliabilities, durability, service life cycle it is necessary to prefix for every building component the performance levels under which it has to be considered in state of damage.

Actually, in the building process for the reasons already underlined :

- on the one hand, it isn't easy to fix objective a measurable or codifycable performance level;
- on the other hand, at least for most building components, the bi-stability does not demand the definition also of intermediate performance level between the initial one and the prefixed minimum one to associate progressive intensities of interventions, as indicated in figure 1.

| DEGRADATION | | PERFORMANCE | | INTERVENTION TYPOLOGY |
|---|---|---|---|---|
| State 1 | ® | Level 1 | ® | Monitoring / inspection |
| State 2 | ® | Level 2 | ® | Cleaning / repair of surface |
| State 3 | ® | Level 3 | ® | Repair / restoration |
| State 4 | ® | Level 4 | ® | Partial substitution / integration |
| State 5 | ® | Level 5 | ® | Total substitution |

**Fig. 1: to any condition of conservation and therefore of degradation a performance level corresponds, which cause a type of maintenance operation, of gradually increasing intensity, in the timing of the general program**

As far as the five indicated typologies of interventions, it is possible to propose the following definitions:

- *Monitoring /inspection*

It is finalized both to control the congruence between plan forecasts and effective in service behaviour and to the sub - condition maintenance strategy. In both cases it is aimed at defining the location of anomalies which can cause imminent compromission of emergency, hygiene, general feasibility.

- *Cleaning / repair of surface*

Epidermic intervention, either because realized on finishing parts or regarding the most superficial layers of no finishing elements; they are non-invasive and to lowest or null technological involvement of other parts;

- *Repair:*

participation aimed at eliminating the anomalies in respect of the initial conditions of the restoration, also when the achieved performances aren't the required ones (however higher than the prefixed minimal level), to execute for extending the average life of the part until the total substitution.

- *Partial substitution / integration*

Intervention in which a part of the element, the subsystem or the system, is removed or replaced because insufficient or superficial repair, in other words when it isn't efficient to eliminate the damage without adding new parts or however by modifying the subsystem or the system.

- *Total substitution:*

It coincides with the "death" of the component and therefore it identifies its life cycle: its total substitution starts a new programming, which will consider the *mid*-life.

The articulated unity of characteristics to be estimated could render in many cases, if lead by sophisticated and complex techniques, extremely complex and onerous the activity of monitoring and diagnosis, above all because of a strong specialized character.

It appears to be necessary to institute a biunivocal correspondence type between the numerical performance data (when it is possible) and their visible manifestations, to interpret as lessening of the value, by causing a noticeable anomaly without particular diagnostic equipments.

In brief, the managerial phase of a maintenance program should be rendered more accessible for the present force-job on the market, with the possibility to characterize the most appropriate intervention to resolve any unplanned situations that are unexpected and undesired.

A solution can be represented by methodologies like the M.E.R. that offers the possibility to execute a diagnosis on the base of detailed descriptions.

This opportunity is possible for some building components (for example, these assumed in the present study), and not for others as some structural elements can be subject to performance failure surveyed by instrumental investigation.

With reference to figure 1, in which 5 "states" are codified that correspondent to as many situations of degradation, for example, it reports a sort of manual for the determination, in phase of planning, of the technology and the chronology of the operations to program, and in managerial phase, for the guide of the designer towards the location of situations to being able to face in the most appropriated way. For each of the assumed building component there are descriptions conceived not to be influenced from the detector. For each one of the "defined states" therefore it will be necessary to characterize the most probable recurrence, to plan the temporary planning, that doesn't account for the mutual ties of technological nature and operating nature between the various parts in which the building organism has subdivided itself.

*2.1.1.2 Coverings of facade*

2.1.1.2.1 with natural stone /marble

state 1 initial similar conditions, absence of anomalies and superficial patina

state 2 presence of superficial patina that alters the chromatism

state 3 incipient flaws and/or separations that interest limited zones (< 10%)

state 4 in action flaws and/or separations that interest limited zones (< 30%)

state 5 in action flaws and/or separations that interest widespread zones (> 30%)

2.1.1.2.2 with bonded materials (grass like, ceramic, etc.)

state 1 initial similar conditions, absence of anomalies and superficial patina

state 2 presence of superficial patina that alters the chromatism

state 3 incipient flaws and/or separations that interest limited zones (< 10%)

state 4 in action flaws and/or separations that interest limited zones (< 30%)

state 5 in action flaws and/or separations that interest widespread zones (> 30%)

2.1.1.2.3 with rended plaster / coloured

state 1 absence of considered alterations

state 2 soot presence

state 3 incipient flaws and/or separations that interest limited zones (< 10%)

state 4 in action flaws and/or separations that interest limited zones (< 30%)

state 5 in action flaws and/or separations that interest widespread zones (> 30%)

*2.1.2.1 external frame in wood/steal*

state 1 absence of considered alterations

state 2 varnish degraded in some points

state 3 varnish degraded on the greater part of the surface; some elements are damage but the frame has still good wear to the air and the water

state 4 the frame is difficult to maneuver, the packings are deteriorated like the varnish and some elements; the wear to the air and the water is partial

state 5 the frame doesn't have wear to the air and the water

*2.1.2.3 external frame in aluminum /PVC*

state 1 absence of anomalies

state 3 frame difficult to maneuver; deteriotated packings

state 5 the frame does not have wear to the air and the water

*2.4.1.1 plane continuous covers*

state 1 absence of discontinuity and infiltrations in below areas

state 2 limited problems of stagnation; degradation of the protecting varnish state 3 deficiency of liquid mortar in the floor bindings if present; limited discontinuities in the impermeable mantle; sporadic breakaway of the folds

state 4 widespread discontinuity and breakaway with problems of infiltrations in below zone; floor elements (if present) not well fixed and/or unconnected

state 5 widespread water infiltrations in below areas.

## 4 MID-NORMAL VALUE AND CASE-STUDY

The mid-normal value $D_{mn}$ of the "duration" of a building component is the statistic value surveyed for some sample buildings.

For this reason, it is preliminarily necessary to characterize these buildings, according to the following criteria:

- quality and quantity of the available information: year of construction, constructive technologies, execute operations, anomalies found during the years, etc;
- possibility to carry out activity of monitoring, to characterize the meaningful variations in terms of performances of the building components;
- characteristic of execute operations: possibility of comparison, degree of innovation, etc;
- homogeneity of the influence agents, whose values constitute those correlated to the mid value, that is those to whose recurrence is from associating value 1;
- possibility to have a variability for single groups of influence agents, in order to be able to estimate correctly the weighting.

For obvious reasons of convenience, we have taken into consideration building-samples of the city of Naples, in reference to two precise constructive typologies:

- buildings with carrying structure in tuff masonry, constructed in the period between the end of the 800 and first decades of the ' 900, known as "class 1";

- buildings with carrying structure in reinforced-concrete constructed in years ' 50/' 60, known as "class 2".

In particular, the articulation of the proposed model suggested the choice of a sub-ensemble of reference for every building in relation to the technological unit "vertical closing", as prospected.

So the prospect to consider mid-normal has the following characteristics:

- exposure to south-west;
- aggressive area;
- free facade (d>>H);
- disposition on the road to intense/fast traffic;
- smooth surface;
- plane vertical surface;
- presence of openings;
- external polish frame (windows);
- vertical external frame;
- practicable and not continuous flat covers with medium slope < 5%;
- practicable and not continuous flat covers with expansion joints;
- practicable continuous flat covers with discontinuity to the extrados;
- common mortar plaster painted with light color;
- prospects of the ancient buildings with cornice, stringcourse, small overhangs and with openings;
- external vertical closings of the ancient buildings in tuff masonry;
- prospects of the modern buildings with cornice, large overhangs and with openings;
- external vertical closings of the modern buildings in brick masonry;
- external frame of the ancient buildings in painted wood;

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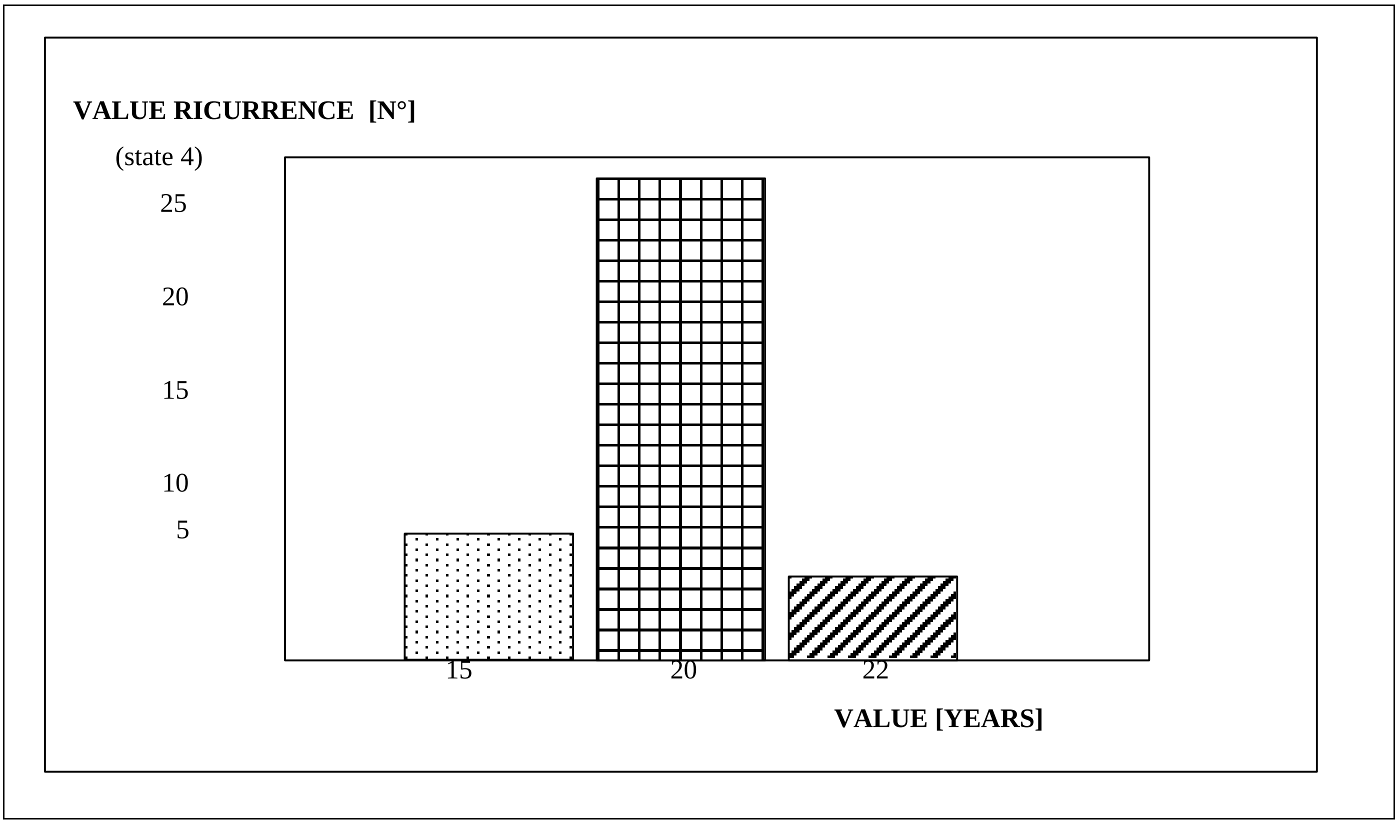
- external frame of the modern buildings in aluminum;
- practicable continuous flat covers with natural covering;
- practicable continuous flat covers with antifreeze covering;
- practicable continuous flat covers with covering put down without the grout;
- not practicable continuous flat covers covered with bituminous girdle armed with polyester with flexibility to cold > -5 °C;
- not practicable continuous flat covers covered with bituminous girdle armed with polyester with flexibility to cold > -5 °C protect with acrylic varnish;
- not practicable continuous flat covers covered with bituminous girdle armed with polyester with flexibility to cold > -5 °C put down on old asphalt.

Following is the histogram of the main collected values. We can see that for 28 times, the value <<<22o years" has appeared (and so it is assumed then effectively which $D_{mn}$ ) on the 48 cases of study regarding the buildings of class 1, while – in any case – the other values are very near to the most recurrent one.




## 5 CHOICE AND DETERMINATION OF THE AGENTS AND OF THE MODIFYING FACTORS.

The choice of the agents and consequently of the relative modifying factors should comply two types of requirements:
- to take in consideration in a systematic way all the agents that influence the reliability of a building component in service, to not commit conceptual errors in the base hypotheses;

- to have a user-friendly instrument, that is able to neglect the influence agents of little significance, tough present.
The criterion to select a set of agents for single type of considered building component, rather than attempt of generalization of the groups of agents, was adopted.

For each of these, subgroups are taken into consuideration which corresponds to a factor; not to render too long and complex the calculation of the product of the mid-normal value for the various factors.
For example, the factors for the external plaster are organized in four groups:

1. CLIMATIC AGENTS
2. ENVIRONMENTAL AGENTS
3. CONFIGURATION AGENTS
4. TECHNOLOGICAL AGENTS

each one sub-divided and later specified in detail.

## 6 ARTICULATION OF THE PROPOSED METHODOLOGY

The hypotheses formulated in the arrangement proposal of the methodology are the following:

a) the 4 groups of agents define 4 modifying factors, which have different weights according to the following proportions:

Climatic 4;

Environmental 4;

Configuration 1;

Technological 2.

b) the exposed values correspond to *the range* of values that the four groups can assume, as a function of their weight in the reliability of the assumed building components:

| | |
|---|---|
| Climatic: | $0.675 \leq F_1 \leq 1.075$ |
| Environmental: | $0.735 \leq F_2 \leq 1.135$ |
| Configuration: | $0.92 \leq F_3 \leq 1.02$ for the coverings |
| | $0.95 \leq F_3 \leq 1.05$ for the fixtures |
| | $0.97 \leq F_3 \leq 1.07$ for the covers |
| Technological: | $0.85 \leq F_4 \leq 1.05$ for the coverings in the buildings of class 1 |
| | $0.875 \leq F_4 \leq 1.075$ for the coverings in the buildings of class 2 |
| | $1 \leq F_4 \leq 1.2$ for the frame in the buildings of class 1 |
| | $0.8 \leq F_4 \leq 1$ for the frame in the buildings of class 2 |
| | $0.87 \leq F_4 \leq 1.07$ for the practicable covers |
| | $0.87 \leq F_4 \leq 1.07$ for the non practicable covers |

c) So the formula for the calculation of $D_{pp}$ assumes the following form:

$D_{pp} = D_{mn} \times F_{CL} \times F_{To} \times F_C \times F_T$

being, respective:

$F_{CL}$ the modifying factor by the climatic agents;

$F_{To}$ the modifying factor by the environmental agents;

$F_C$ the modifying factor by the configuration agents;

$F_T$ the modifying factor by the technological agents;

d) For the calculation of single Fi the following procedure is adopted. On the base of the determined scores (that are inversely proportional to the ability to produce situations of reliability), every 4 group of factors has *a range* of possible values between the minimum and the maximum. Inside this *range* there is the mid-normal value (than can't be in the center of the range) to which corresponds modifying factor 1; the values comprised between this value and the minimum will assume values smaller than the unit (they provoke lessening of the reliability), those comprised between this value and the maximum (they correspondent to better conditions for the reliability) will have values greater than the unit.

e) To determine the value of the factor for a group of agents, it is possible to operate as follows:

- defined $\Delta F$ the refuse of factor (point b);

- defined $\Delta P$ the refuse between the maximum score and minimum;

- defined $P_{mn}$ the value of the total score correspondent to the situation defined as mid-normal (factor 1);

- defined $P_{max}$ and $P_{min}$ the scores maximum and minimum;

the value of V to adopt will be comprised between the following values:

$V_{min} = 1 - [\Delta F / \Delta P \times (P_{mn} - P_{min})]$

$V_{max} = 1 + [\Delta F / \Delta P \times (P_{max} - P_{mn})]$

generically:

$V = 1 + [\Delta F / \Delta P \times |P_{mn} - P|]$

## 7 INFLUENCE AGENTS FOR THE EXTERNAL PLASTER.

The groups of influence agents for the assumed component are the following:

1. *CLIMATIC*

1.1 TEMPERATURE
1.2 DAILY ΔT TEMPERATURE
1.3 WIND AND RAIN
1.4 SNOW
1.5 MOISTURE

2. *ENVIRONMENTAL*

2.1 POINTS CARDINALS
2.2 SEA
2.3 POLLUTION
2.4 VIBRATIONS
2.5 OTHER BUILDINGS FACING

3. *CONFIGURATION*

3.1 SUPERFICIAL ASPECT
3.2 LYING
3.3 SHAPE/NUMBERS OF ANGLES
3.4 EXTENSION

4. *TECHNOLOGICAL*

4.1 PROTECTIVE ELEMENTS
4.2 CRITICAL POINTS
4.3 COLOR
4.4 KIND OF STRINGING AND NATURE OF THE SUPPORT

## 8 TREND OF THE RESEARCH

The developed software manages numerical data from the experimentation executed on the field.

Better examined in other paper [1] a laboratory experiment is now starting, in which the data of tests will be backed up in order to equal the modifying factor.

In this sense, there won't be any problems of correlation between experimental data and test data or any problems of re-scaling, because the laboratory will provide reliable assessment of the relative incidence of the Fi, and not on absolute incidence, that would give problems for the comparison with results from experimentation executed on the field.

In parallel, another research has started. This research is aimed at exporting to other building components the study based on the external plaster: for example, the EBR FRP (*Fiber Reinforced Plastics Externally Bonded Reinforcement)* [2] .

Another phase of test is developing regarding the model already implemented, through a reading and a verification of back data collected for external plaster in others and various areas of the Italian territory.

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# The Factor Method For Service Life Prediction From Theoretical Evaluation To Practical Implementation

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**Summary**: **This paper presents a brief summary of a state-of-the-art-report which has been worked out within CIB W80/RILEM 175-SLM "Service life methodologies" on the so called Factor Method for service life prediction of building materials and components. The Factor Method was presented in ISO 15686 Part 1, and has gained considerable interest as a simple tool for service life prediction of building materials and components. The paper contains a description of expressed needs for service life prediction methods, requirements which have been described for such methods, description of the Factor Method itself and a presentation of several theoretical and practical evaluations of the Factor Method. Finally, it shows some needs for further development if the method is going to be applied as a practical tool in the different phases of a building structure.**

**Keywords. Service life, prediction method, Factor Method, ISO 15686 Part 1.**

## 1 INTRODUCTION

Buildings and constructed assets constitute a major part of the real value of a country. The planning, construction, use, management and demolition of buildings and constructed assets therefore play a very important role in the development of more sustainable societies. And an important part of this development are the efforts to obtain optimal service lives of the buildings and constructed assets.

Knowledge about service life of building materials, components and buildings has been developed in many countries for many years, and different kinds of tools for service life prediction have been established. During the last decade, international standardisation of service life prediction tools has taken place by the International Organization for Standardization (ISO). The actual subcommittee being responsible for this work is SC 14 “Design life”. Most of the development in advance of the standardisation takes place within working groups.

In 2000, ISO published a standard ISO 15686 Part 1 (ISO 2000), which is the first part of a series of standards dealing with service life of buildings and constructed assets. As a part of the standard is a description of a service life prediction method, the socalled Factor Method. This is presented as a simple, deterministic method which should be very simple in use, provided that the necessary input data are available. The method is based on similar factorial methods which have been developed in Japan (Principal Guide 1993), and it has been under discussion and evaluation for several years within the international committee CIB W80/RILEM 175-SLM “Service life methodologies”. Most of the discussion and evaluation has been on a theoretical basis, and so far there has been limited experience in using the method in practice. However, there is a great interest in the method, and during the last few years some practical examples in trying to apply and evaluate the method have been published.

## 2 NEEDS FOR SERVICE LIFE PREDICTION METHODS

Over the last decades, there has been an increasing focus on the needs to determine durability and service life of materials, components, installations, structures and buildings. This has been based on two important aspects:

- **environmental issues**; lack of material and energy resources and the building and construction sector as a big consumer of these resources, and the environmental impact caused by buildings.
- **economical issues**; the total value of the built environment on a national level and the value of each specific unit of it (buildings, structures, roads, bridges, quays, etc.) for the specific owner (authorities, private companies or individuals). The conditions of the built environment, the annual costs of management and maintenance and the life cycle costs are of major importance for the economy and the competition.

The importance of these aspects is reflected in several initiatives and activities at both the international and the national level. Many of the activities have ended in publishing regulations or standards, and in these documents the needs for service life prediction of building products and components are stated. Most of them are describing requirements at a national or regional level, but at the same time they reflect an important and increasing trend to focus on this issue. Some of the specific requirements that have been published are briefly mentioned below, as an illustration of how they are expressed.

In Europe, the Construction Products Directive (CPD 1988) has now been implemented in the European Economic Area (EEA) countries. In the CPD it is stated that any construction product which is covered by the CPD, shall have such properties that the building or structure is able to fulfill six specific essential requirements. The requirements shall be fulfilled during an economically reasonable working life of the products. The term working life is corresponding to service life. Each of the six essential requirements are explained more in detail in six corresponding Essential Requirements (Essential Requirements 1994), and these documents also contain a specification of what is meant by working life and how to take care of durability issues for the construction products. The following explanations are given for the working life in all the Interpretative Documents:

***"1.3.5 Economically reasonable working life:***

*(1) The working life is the period of time during which the performance of the works will be maintained at a level compatible with the fulfillment of the essential requirements."*

The Construction Products Directive is now a basis for introduction of performance based building regulations in European countries, and thereby requirements for durability and service life of construction products are implemented into national building regulations in Europe.

In 1992, a new building code was published in New Zealand (NZBC 1992) which contains quantitative requirements for the service life of various parts of buildings or for construction products. The clause B2 Durability contains specific requirements both in a qualitative and a quantitative formate.

In the Canadian Standard CSA 478-95 (CSA 1995), a description is given of the relation between design life of a building or a building component, and the durability of the component. The requirements for durability are expressed in terms of design service life, and typical dersign service life categories for buildings are given in the standard.

In 1999, the European Union (EU 1999) published a Guidance Paper containing a table of assumed working lives of works and construction products. The table is shown in Table 1. The table has also been published by the European Organization for Technical Approvals (EOTA) (EOTA 1999), and it is another example of how quantitative values are given for service life which architects, consultants, authorities and manufacturers of building products have to take into consideration and be able to fulfill.

**Table 1 Design lives of works and construction products for various categories of buildings. (From EOTA (1999))**

| Assumed working life of works (years) | | Assumed working life of construction products (years) | | |
|---|---|---|---|---|
| Category | Years | *Category* | | |
| | | Repairable or easily replaceable | Less easily repairable or replaceable | Lifetime of works ** |
| **Short** | 10 | 10 * | 10 | 10 |
| **Medium** | 25 | 10 * | 25 | 25 |
| **Normal** | 50 | 10 * | 25 | 50 |
| **Long** | 100 | 10 * | 25 | 100 |

* In exceptional and justified cases, e.g. certain repair products, a working life of 3 or 6 years may be envisaged.

** Products not repairable or economically replaceable.

In a similar way, recommendation for minimum design life of a building or building components are given in the international standard ISO 15686 Part 1 (ISO 2000). This recommendation is shown in table 2.

**Table 2. Suggested minimum design lives for components. (From ISO (2000)).**

| Design life of building | Inaccesible or structural components | Components where replacement is expensive or difficult (incl. below ground drainage) | Major replaceable components | Building services |
|---|---|---|---|---|
| Unlimited | Unlimited | 100 | 40 | 25 |
| 150 | 150 | 100 | 40 | 25 |
| 100 | 100 | 100 | 40 | 25 |
| 60 | 60 | 60 | 40 | 25 |
| 25 | 25 | 25 | 25 | 25 |
| 15 | 15 | 15 | 15 | 15 |
| 10 | 10 | 10 | 10 | 10 |
| NOTE 1: Easy to replace components may have design lives of 3 or 6 years | | | | |
| NOTE 2: An unlimited design life should very rarely be used, as it significantly reduces design options. | | | | |

## 3 REQUIREMENTS FOR SERVICE LIFE PREDICTION METHODS

As mentioned in the Introduction, there has been much evaluation and discussion of service life prediction methods over the years, and mostly on a theoretical basis. As specific service life prediction methods have been published and the different users have got some experience, the requirements have been more specific.

Already in 1987, Masters (1987) expressed some general requirements to a service life prediction system in the way of ten commandments. In documents describing durability and service life prediction methods (Principal Guide 1993, BS 1992, CSA 1995, Sarja and Vesikari 1996, ISO 2000), there are also recommendations and explanations of what is needed both as input for the use of the methods and for a safe and reliable evaluation and use of the outcome. In many of the documents it is stressed that prediction of durability and service life is subject to many variables and can not be an exact science. Such variables are material quality, environmental conditions, installation, operating and maintenance procedures. The results have to be treated as an indication of what will be the service life, when taking the actual factors and circumstances influencing the durability and service life into consideration.

Martin et al. (1995) have carried out a comprehensive study on methodologies for predicting the service life of coating systems. They present a set of criteria for judging the adequacy of any proposed service life prediction methodology. These criteria include the ability to:

1. Handle large variability in the times-to-failure for nominally identical specimens
2. Analyze multivariate data
3. Discriminate among these variables. That is, the service life prediction methodology should be able to separate the few significant variables from the many insignificant variables
4. Fit both empirical and mechanistic failure models to short-term laboratory-based exposure results
5. Establish a connection between short-term laboratory-based and long-term in-service results
6. Provide mathematical techniques to predict the service life of a coating system exposed in its intended in-service environment

Finally, we should mention a discussion paper by Bourke and Davies (1997), where a list of essential and/or desirable characteristics of a service life prediction system is presented. They state that

*"the relative importance of each is arguable, but important features may be considered to include the following:*

- easy to learn
- easy to use
- quick to use
- accurate

- easy to update
- easy to communicate
- adaptable
- supported by data
- links with existing design methods and tools
- free of excessive bureaucracy
- recognises the importance of innovation
- relevant to diverse environments
- acceptable to practitioners and clients alike
- reflects current knowledge
- a flexible level of sophistication for either outline or detailed planning"

## 4 DESCRIPTION OF THE FACTOR METHOD

The Factor Method as described in ISO 15686 Part 1 (ISO 2000) is based on a variety of factorial methods being developed in Japan (Principal Guide 1993). One one hand, the ISO Factor Method represents a simplification compared to the Japanese methods. On the other hand, this simplification gives less opportunity to take care of important issues as material used, special climatic conditions and other circumstances.

A factorial method for evaluation of surface treatment of wooden windows and doors has been developed in Germany (Verband 1997). This method is used to evaluate the service life of the products, based on information about the material and surface treatment used and on the exposure conditions. The method has been developed independantly of the ISO Factor Method.

The ISO Factor Method is presented in the standard in the following way:

> ***"9. Factor method for estimating service life***
>
> ***9.1 Outline of the factor method***
>
> *The method allows an estimate of the service life to be made for a particular component or assembly in specific conditions. It is based on a reference service life (normally the expected service life in a well-defined set of in-use conditions that apply to that type of component or assembly) and a series of modifying factors that relate to the specific conditions of the case.*
>
> *The method uses modifying factors for each of the following:*
>
> *- factor A: quality of components*
>
> *- factor B: design level*
>
> *- factor C: work execution level*
>
> *- factor D: indoor environment*
>
> *- factor E: outdoor environment*
>
> *- factor F: in-use conditions*
>
> *- factor G: maintenance level*
>
> *Any one (or any combination) of these variables can affect the service life. The factor method can therefore be expressed as a formula:*
>
> *ESLC = RSLC x factor A x factor B x factor C x factor D x factor E x factor F x factor G."*

The reference service life of a component (RSLC) is defined as

> *"service life that a building or parts of a building would expect (or is predicted to have) in a certain set (reference set) of in-use conditions."*

In the standard there is also a brief discussion of the use of the Factor Method, and a discussion of the reference service life as well as each of the modifying factors.

## 5 EVALUATION OF THE FACTOR METHOD

Since the introduction of the Factor Method in the first draft of the ISO Standard, it has been evaluated in several papers, both on a theoretical basis as well as based on some practical appliactions. In this paragraph we will refer to some of the theoretical evaluations, whereas some examples of practical application and evaluation are presented in the next paragraph.

The Standard ISO 15686 Part 1 (ISO 2000) itself contains a chapter which describes the Factor Method. In that chapter, there is a general discussion as well as a discussion of the reference service life (RSL) and each of the factors. In the general comments of the Factor Method (chapter 9.2), it is said that

> *"The factor method is a way of bringing together consideration of each of the variables that is likely to affect service life. It can be used to make a systematic assessment even when reference conditions do not fully match the anticipated conditions of use. Its use can bring together the experience of designers, observations, intentions of managers, and manufacturers' assurances as well as data from test houses.*
>
> *The factor method does not provide an assurance of a service life: it merely gives an empirical estimate based on what information is available. It is different from a fully developed prediction of service life, which will ideally provide the reference service life for a factored estimate."*

A thorough discussion of the Factor Method has been presented by Bourke and Davies (1997). The report is intended to give a contribution to the further development of the method described in (ISO 2000). In the general summary and conclusions of the report, the authors state that

> *"The system would serve initially as a means of permitting objective comparison and analysis rather than as a firm prediction of anticipated years on service.*
>
> *This however should not disguise that the effect of adoption of such a system should be to optimise the selection of components, making large-scale, expensive and disruptive remediation unnecessary. Equally, excessively durable specifications for short-life buildings could be reduced. It would also highlight the ease with which durability could be improved "on the drawing board", thereby achieving enhanced performance for minimal cost."*

Hovde (1998) has presented an evaluation of the Factor Method. It is based on considerations and discussions e.g. within CIB W80/RILEM 175-SLM. In the short range, there is a need for input data both for the quantification of the reference service life (RSL) as well as for the different factors in the equation. In the long range, there will be a need for a more comprehensive evaluation of the Factor Method, including possibilities of quantitative description of the RSL and the factors. Hovde also pointed out that the method should be evaluated according to the general requirements for service life prediction methods, like the requirements mentioned above. He gives a brief discussion of the following items which ought to be further evaluated:

- estimation of the reference service life (RSL)
- important factors
- necessary number and type of factors
- use of the factors in an equation
- reasonable span of the values of the different factors
- relative importance of the factors
- uncertainty of the factors
- factor dependency on material or component to be evaluated
- important considerations for practical use

The last item is illustrated in figure 1.

Aarseth and Hovde (1999) and Moser (1999) have tried to further develop the Factor Method by including a probabilistic approach in the selection of the factor values. Hovde and Aarseth are applying a "step-by-step" principle where the value of each factor is given by a triple estimate, i.e. a minimum, most expected and maximum value. In order to give a reasonably good statistical representation of the triple estimates, an Erlang density function is used. The modified Factor Method has been applied to estimate the service life of a wooden window, based on one of the examples given in (ISO 2000). In the example, factor values are chosen so that the estimated service life (ESC) becomes 62.2 years, i.e. 60 years. By applying the same factor values as the most expected values and guessing minimum and maximum values, the estimated service life becomes 50 ± 6 years. In the conclusions, the authors state that

> *"The "step-by-step" principle enables a stochastic handling of the modifying factors in the ISO factor method by performing a triple estimate for each factor. After the statistical calculation the estimated service life is expressed as three figures: the exptected value plus/minus one standard deviation.*

*The "step-by-step" principle also demonstrates a systematic approach to the estimating process, including the process of defining, subdividing and estimating the modifying factors in the ISO factor method."*

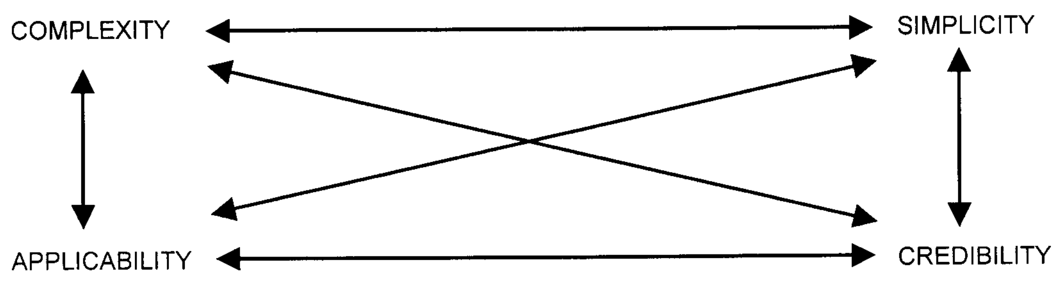


**Figure 1. Relations to take into consideration in evaluation of the Factor Method. (From Hovde 1998)**

Moser (1999) has applied an individual statistical treatment of each factor. This is done by using different statistical distributions for each factor (like deterministic, normal, lognormal or Gumbel), and by giving individual figures for the minimum, most probable and maximum value of each factor. In his conclusions, Moser states that

> *"The use of probabilistic tools for planning maintenance or replacement costs is no longer only restricted to projects having large funding requirements or numerous assets. Making full use of the information, e.g. given in ISO/CD 15686 and modified by professional opinion, permits the use of variables instead of deterministic factors in the equation for the estimated service life. The results give a much more detailed insight into the service life of building components involved and allow a far better planning of the investments required."*

Other evaluations and discussions of the Factor Method have been published by Lounis et.al. (1998), Teplý (1999) and Rudbeck (1999).

## 6 PRACTICAL USE OF THE FACTOR METHOD

As mentioned earlier, the Factor Method has attained considerable interest during the development stages, and several researchers have tried to apply it for practical purposes. This has given very valuable results and experience, and such practical application is an important basis for further development and improvement of the method. Most of the public cases are described in research papers or reports where the application is shown as examples. The practical application of the Factor Method has been limited due to little knowledge of the method among practitioners as architects, consultants or building owners and managers, or due to lack of input data for the various factors of the method.

There is also an extensive research and development going on regarding design for durability of buildings, sustainable construction and development of sustainable buildings. Several papers have been published during the last years describing different challenges in this area, and they show the necessity of implementing service life of materials and components into the overall methods. Wyatt and Lucchini (Wyatt 1998, Lucchini and Wyatt 1999, Wyatt and Lucchini 1999) have published several papers regarding design and construction of sustainable buildings. In Wyatt and Lucchini (1999) they conclude that

> *"Whilst there is a complex but important relationship between the building performance the designer may have pre-defined and the level of reliability achieved one cannot guarantee a building's life future. One can however, be prudent and accept the place of a service life practice and life care to at least seek a building product life.*
>
> *So adopting the service life approach that reflects materials, components and systems service loss and durability would help to improve cost certainty of building ownership and begin to address the challenges posed in striving for sustainable cities and buildings. For it has become clear that designing for durability has an immensely important contribution to make to both the work of CIB W94 and CIB W80 as well as ISO task group responsible for developing the Design Life of Building's Standard.*
>
> *It is now believed that service life and its practice will come to form and be seen as the corner stone of building asset management's life care."*

Strand and Hovde (1999) have carried out at study of how service life data of exterior surface materials (wood and brick) influence on the life cycle assessment (LCA) of the materials. The authors wanted to show how service life data are needed in LCA, how the data occur and how they influence the results. Building materials and components are used for a longer period of time than most other products. LCA of a building product therefore necessitates gathering of data that will be valid for a longer period of time. The authors apply the Factor Method as described in ISO 15686 Part 1, but with main emphasis on the factors E (outdoor environment) and G (maintenance level). LCA is carried out for two climates (industrial and rural inland) and for facades facing north or south. Different intervals for painting, cleaning and replacement are also used.

Hovde (1999) has presented the need for service life prediction of passive fire protection systems (fire retardants). He refers to the Factor Method as described in ISO 15686 Part 1. Passive fire protection has got an increasing interest and importance in relation to the introduction of performance based building and fire codes. This makes it important to predict the durability and service life of the fire protection systems, and this will be a specific area for application of service life prediction methods.

A joint RILEM and CIB committee (RILEM TC 172-EDM/CIB TG 22 “Environmental design methods in materials and structural engineering”) has been working on development of methods for environmental design of materials and structures. A progress report of the work has been presented by Sarja et al. (1999). In the presentation of the progress report it is explained that the incorporation of an environmental viewpoint into the design of materials and structures, it is necessary to reconsider the entire context of the design process in order to integrate environmental aspects into a set of other design aspects. Further, this kind of process is called integrated life cycle design, and it it is said that the aim of the process consists of assimilating, in a practical manner, the multiple requirements of functionality, economy, performance, resistance, aesthetics and ecology all into the technical specifications and detailed designs of materials and structures.

The joint RILEM/CIB committee is producing a manual which will provide methods and methodologies for structural design in order to meet the requirements of sustainable development over the entire service life of the structures. The scope of the manual includes both bearing and non-bearing structures of buildings, bridges, towers, dams and other structural facilities. In the description of the design process in the progress report, there is also a presentation of alternative methods which can be applied for durability design.

In the conclusions of the progress report, it is stated that

> *“Concerning materials and structures, new basic knowledge will be needed especially regarding environmental impacts, hygrothermal behaviour, durability and service life of materials and structures in varying environments. Structural design methods that are capable of life cycle design, multiple analysis decision-making and optimisation will have to be further developed. Recycling design and technology demand further research in design systematics, recycling materials and structural engineering. The knowledge obtained will have to be put into practice through standards and practical guides.”*

Hed (2000) has carried out a study of service life planning for a multi family building, which was built in Gävle, Sweden, in 1999. ISO 15686 Part 1 was used as a basis for the study. The service life planning was integrated into the design of the building and followed the building process from the design phase to the beginning of the construction of the building. The report comprises three separate papers, and in one of the papers are given a presentation and discussion of the application of the Factor Method as presented in ISO 15686 Part 1. The author states that

> *“A problem is that there are still few tests performed of material and component service life, comprising all the effects required of the building component when it is operation in the building, i.e. following the service life prediction methodology (ISO 1999).*
>
> *The accuracy of the estimated service life is of course suffering from this fact, so one has to discuss if it is worth the effort of doing the estimations or not. If the goal is to find a precise value it is clear that the goal is not reached. But if the goal is to improve the general situation in service life planning the answer is yes.*
>
> *The factor method in ISO/DIS 15686-1 is meant to be a tool to improve the estimation of the service life. It was found in the project that this method did not improve service life estimations. This opinion is summarised in the following.*
>
> *Uncertainty of RSLC and values of Factors. The factorial formula (1) comprises in the right side of a reference value (RSLC) and the adjusting Factors, A to G. If the reference value can not be determined accurately it is not appropriate to adjust these values with a set of uncertain Factors.*
>
> *Uncertainty of the effect by combination of Factors. The method does not support the thoughts that one needs knowledge of cause and effect to estimate the service life. The estimation will be based on uncontrollable occurences, which can act independently of each other.”*

In a Nordic Research Training Course funded by the Nordic Academy for Advanced Study (NorFA) which was carried out during 2001 regarding service life of buildings and building products, some of the participants gained interest in the Factor Method as a simple tool for service life prediction. The title of the course was "Service life of buildings - from theory to practice", and the research students carried out individual project tasks related to the main topic of the course. The project reports are at a preliminary stage, and the intention is that they will form the basis for papers to be published in journals, conferences, etc. In the reports are discussed the possibilities to apply the Factor Method for service life prediction of different building products, components, structures and installations, such as surface products, exterior wood products, external renderings, sulfur concrete, a solar collector and a fibre reinforced polyester pedestrian bridge deck. This clearly underlines the fact that there is a need for practical and simple service life prediction tools, and that the Factor Method may be evaluated for practical use on a wide variety of products in the future.

One of the reports from the Research Training Course is mentioned here. Marteinsson (2001) has presented the results of a big condition survey of wooden windows in Iceland. He has applied the Factor Method to study the service life of the windows. In the report the results from the condition assessment and the house owners' answers to a questionnaire are combined, and the Weibull probability distribution is then used to evaluate the estimated service life of the windows. In the conclusions of the study, Marteinsson states that

*"The results show that for some materials at least, the synergy between agents that affect the durability of materials is so great that it is difficult to give each and one of the factors in the standard a value even based on results gained by systematic study of the object in use. The results show furthermore that for materials where the durability gains very much from good careing and maintenance, then it is a good way to decide on the probability distribution of the service life from information from the user.*

*The realistic span in multiplication factor is thus considerable and the user of the methodology will not be able to choose appropriate values for the factors without extensive knowledge about materials and local building practice. In any case he needs an information about the main factors for the component and material considered, (and) what span is normal for the factors. The methodology is far from easy to use correctly and at the risk of results being evalulated as being more precise than is reasonable at the present time.*

*Handbook of worked examples would be of great value for the designer, without this freedom for each user to define the factors is too great."*

## 7 FURTHER DEVELOPMENT AND PRACTICAL APPLICATION OF THE FACTOR METHOD

The brief state-of-the-art of the Factor Method given above has underlined the fact that service life prediction of buildings or building elements, components or products in general is rather difficult and laborious. It is not an exact science, and the service life is dependent on various factors which also make a complete service life prediction an interdisciplinary activity. Service life prediction can be based on two different principal approaches:

- deterministic approach
- probabilistic approach

This gives the basis for development of service life prediction methods of various complexity and with different requirements of applicability and needs for input information. Three levels of service life prediction methods can be described as shown in figure 2.

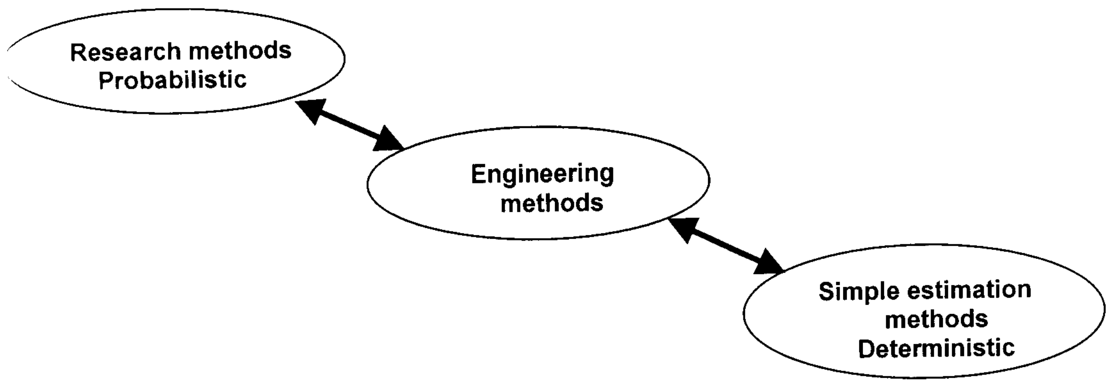


**Figure 2. Relation between different types of service life prediction methods.**

The Factor Method which is discussed in this paper is based on the deterministic approach.

The outcome of the theoretical and practical studies and evaluations described above should be a good basis for further development and application of such methods. The presentation also shows that there is a need for service life prediction methods in many areas, and that the Factor Method is regarded as a simple and easily accessible tool. However, there seems to be still many topics that have to be evaluated further before the method will come in to a wider practical application . The following topics will be of importance:

- determination and collection of data for the reference service life (RSL) and the individual factors
- development of a sound engineering method which combines the benefits of more sophisticated probabilistic methods and simple deterministic methods
- practical use of the method in case studies of specific building materials and components or of specific buildings
- application of the method in life cycle analysis of building materials and components and environmental evaluation methods for buildings
- application of the method in integrated life cycle design and design for durability of buildings

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# Performance-Based Covercrete Concept

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**Summary: There has been a popular move toward performance-based building codes and specifications. Greater emphasis on service life has also led to a strong demand for performance-based durability design principles and specifications. The *quality* and *quantity* of cover concrete (covercrete) are the critical parameters governing the protection of steel reinforcement against external aggressive agents. In particular, reinforced concrete exposed to marine conditions or de-icing salts can be susceptible to chloride-induced corrosion of the reinforcing steel. In this paper, the concept of equivalent covercrete is introduced under the principle that a set of equivalent covercretes will offer the same degree of protection to the steel reinforcement against corrosion. Hence, the same service life should result. By assuming that diffusion is the primary transportation mechanism of chloride ions into the covercrete, and accepting the approximations involved in using the solution to Fick's second law of diffusion, two theoretical sets of conditions are found necessary for equivalent covercrete to be obtained from shorter term chloride penetration data. Firstly, the range of concrete considered must have similar surface chloride concentrations, and secondly consistent ratios of short-term to long-term chloride diffusion coefficients. Analysis of chloride penetration profiles of a series of concrete specimens, exposed to laboratory-simulated exposure for up to five years, suggested that both these conditions can be reasonably satisfied for concretes of grade 40 and above. This provides a basis for equivalent covercrete to be determined and used in modern performance-based specifications.**

**Keywords. Covercrete, design life, marine conditions, chloride ion penetration, AS 3600.**

## 1 INTRODUCTION - CHARACTERISING 'COVERCRETE'

The *quality* and *quantity* of cover concrete (covercrete) are the critical parameters governing the protection of steel reinforcement from aggressive environment. In a number of National Standards for concrete structures, such as the Australian Standard AS 3600 (Standards Australia 1994) and British Standard BS 8110 (British Standards Institution 1985), this concept is clearly illustrated. Many other specifications, however, divorce the requirement of *quality* completely from *quantity*. This weakness must be urgently addressed.

AS 3600 recommends a range of concrete grades (quality) with corresponding cover depths (quantity) for a range of environmental exposures. They are apparently given on the basis that concrete structures built to the recommended specification will have a *design life* of 40–60 years. For structures permanently submerged in sea water (exposure classification B2), for example, covercretes given as [strength grade in MPa, cover in millimetre] of [50, 25] or [40, 35] or [32, 50] are recommended for rigid formwork and intense compaction. In this Standard, the possible variation of *quality* of cover is taken into consideration in terms of the strength grade, i.e. the characteristic strength concept. However, only some tolerances are given for the *quantity* of cover given as absolute minimum covers.

In AS 3600 and its commentary, no information is given on the derivation of these equivalent covercretes. The interrelationship between the *quality* and *quantity* of covercrete with respect to a given design life of concrete in a marine exposure will be examined in this paper.

## 2 DEFINING EQUIVALENT COVERCRETE

The concept of 'equivalent' covercrete is one where a range of covers provides the same degree of protection to the steel reinforcement against corrosion. In marine exposure or structures exposed to de-icing salt, the cause of depassivation and corrosion of steel reinforcement is due to the ingress of chloride through the cover concrete. A *design life* or *service life* has been defined (Cao *et al*. 1998) in a number of ways. Both the 'initiation' model and the 'acceptable corrosion' model are based on chloride penetration into the concrete reaching particular critical levels at the steel reinforcement.

Equivalent covercretes for the same design life ($T$) refer to a set of concretes with varying quality, but with an 'exact set' of corresponding cover depths. These covercretes permit chloride penetration reaching a critical level. The threshold level for corrosion initiation is a subject of intense debate. Cao *et al.* (1998) recommended a critical level of around 0.2% by weight of concrete at the steel at time $T$ if the acceptable corrosion concept was used. This is illustrated in Fig. 1.

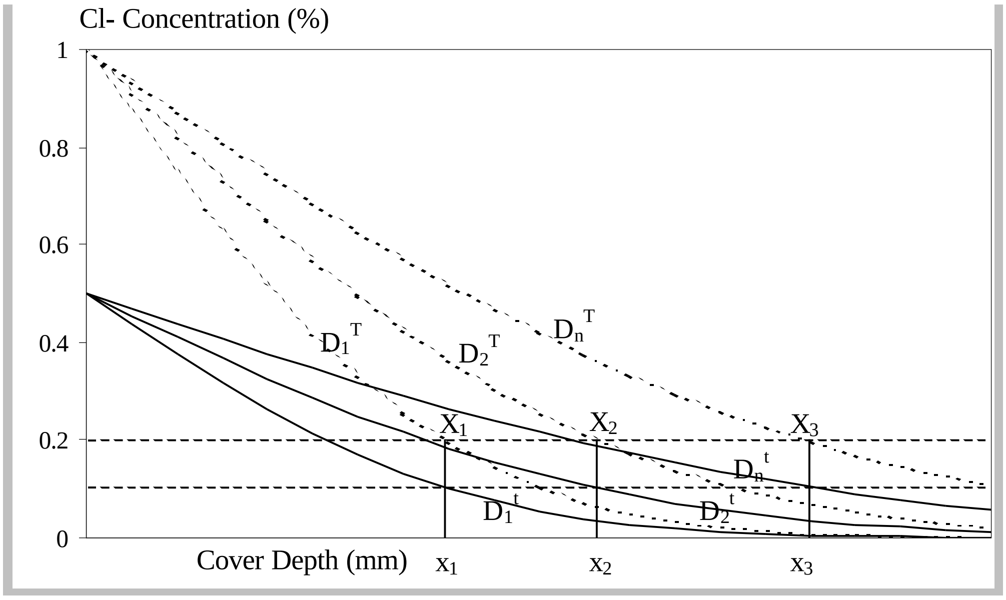


**Figure 1. Concept of equivalent covercrete with a range of concrete of different diffusion characteristics but with the same chloride concentration at $X_1$, $X_2$ and $X_3$**

In Fig. 1, a set of concretes is given whose quality is characterised by their chloride penetration profiles at the design life of time $T$. The range of concretes of interest is of different quality, reflected in different chloride penetration profiles. Their surface chloride concentrations are assumed to be the same value of $C_s$, but with various diffusion coefficients of $D_1^T$, $D_2^T$ and $D_n^T$. At a corresponding cover of $X_1$, $X_2$ and $X_n$, the chloride level is the same. This defines the equivalent covercrete of [$D_1^T$, $X_1$], [$D_2^T$, $X_2$] and [$D_n^T$, $X_\mathrm{n}$].

## 3 MODELLING CHLORIDE PENETRATION INTO COVERCRETE

The determination of chloride profile is possibly the most popular tool used to assess the chloride resistance of concrete, both qualitatively and quantitatively. In qualitative terms, comparisons are made after a similar exposure period of various concrete samples. It is assumed that diffusion is the main transportation mechanism of chloride ions into the concrete. According to Fick's second law, a diffusion coefficient can be calculated by fitting an error function to the measured chloride profile. It must be noted, however, that a chloride diffusion coefficient, determined from a measured chloride profile, is not unique in value. The coefficient is termed an 'apparent chloride diffusion coefficient ($D_a$')'. It can vary significantly with the periods of exposure. This will be further discussed.

Many researchers have successfully developed 'migration' or 'concentration difference diffusion' testing techniques to measure the steady-state chloride diffusivity of concrete. The chloride concentration gradient, obtained from the measured concentrations of the source and receiving chambers, enables the calculation of the diffusion coefficient. Some of these techniques are described by Dhir *et al.* (1990) and Gjørv (1996). Andrade and Sanjuán (1994) outlined an experimental procedure for the calculation of chloride diffusion coefficients in concrete from migration tests. They based their calculations on the theoretical approach using the Nernst–Planck equation which models mass transport in electrolytes, and the Nernst–Einstein equation using electrolyte equivalent conductivity as the main parameter.

### 3.1 Diffusion of non-ionic molecules

Fick's first law of diffusion or steady-state diffusion law is:

$$J = -D_e \, \mathrm{d}C/\mathrm{d}x \qquad (1)$$

Measurement of the diffusion coefficient involves the concentration difference at any two particular planes at $x$ and $x + dx$. The coefficient represents the net chloride flow through concrete. It is referred to as an effective diffusion coefficient $D_e$.

Fick's second law of diffusion or unsteady-state diffusion law states that:

$$\frac{\partial C_t(x,t)}{\partial t} = -D_a \frac{\partial^2 C_t(x,t)}{\partial x^2} \qquad (2)$$

where $C_t$ denotes the total concentration of chloride ions, and $D_a$ is assumed to be constant. The solution for a semi-infinite medium with initial conditions of $C_t$ ($x$ > 0,0) = 0 and boundary condition $C_t$ (0,$t$) = $C_s$ = constant is described by:

$$C_X = C_s - (C_s - C_i)\, erf\left(\frac{X}{\sqrt{4Dt}}\right) \tag{3}$$

where *erf* denotes the error function.  This solution is applicable for the unsteady flow condition where the fluid properties at any point may change with time.

### 3.2 Ionic diffusion

Equations (1) to (3) are valid only for a non-ionic diffusant. In an ionic solution such as chloride solution, they do not strictly apply. It has been shown by Chatterji (1994a; 1994b) that Fick's laws of diffusion need modifications before they can be applied to cement-based materials. The most important modifications are those of Nernst and Nernst–Planck retardations of ion mobility, the formation of the electrical double layers near negatively charged cement hydration products, and the chemical reactions between the migrating ions and cement-based materials.

### 3.3 An engineering solution

In practice, an apparent diffusion coefficient, $D_a$, calculated from Eq. (3) is commonly used to characterise a particular chloride profile. Three approximations are made. Firstly, the chloride profile of the concrete examined is assumed to be obtained from unsteady-state conditions. Secondly, the ionic nature of the chloride in sea water is ignored. Thirdly, the chloride profile is often based on the determination of water-soluble chloride.  $D_a$ obtained with this approximation can be adequately used to characterise the profile but cannot be directly used to predict service life.

The total concentration is measured as acid-soluble chloride ions. $D_a$ therefore takes into account the chemical reactions within the concrete. For example, chloride ions may react with $C_3A$ to form calcium chloro-aluminates. The reaction or adsorption of chlorides by cement pastes is considered by some (Andrade 1993) to be a minor influence, although a number of researchers (Tritthart 1998; Hachani *et al*. 1991) do take it into account. The free chloride ion, measured as water-soluble chloride, is however recognised to be responsible for the degradation of concrete.  The use of water-soluble chloride as total chloride is a convenient approximation adopted in the work reported in this paper.

## 4 Using Short-term Property in Equivalent Covercrete

The concept of equivalent covercrete, described above, will not be very useful in specification. This is due to the lack of long-term properties such as the chloride penetration profile at the design life or the diffusion coefficient of the profile. There is very limited data on the relationship between short-term and long-term properties of the same concretes. In this section, conditions are explored whereby short-term properties may be used to derive a set of equivalent covercrete.

### 4.1 Conditional requirements

For a range of concretes, characterised by long-term chloride penetration profiles and corresponding diffusion coefficients $D_1^T$, $D_2^T$ and $D_n^T$, the level of chloride at each cover depth $X_1$, $X_2$ and $X_n$ is calculated from Eq. (3) for the concretes as follows:

$$C_{X1} = C_s - (C_s - C_i)\, erf\left(\frac{X_1}{\sqrt{4D_1^T T}}\right) \tag{3.1}$$

$$C_{X2} = C_s - (C_s - C_i)\, erf\left(\frac{X_2}{\sqrt{4D_3^T T}}\right) \tag{3.2}$$

$$C_{Xn} = C_s - (C_s - C_i)\, erf\left(\frac{Xn}{\sqrt{4D_n^T T}}\right) \tag{3.n}$$

For a set of equivalent covercrete which gives the same design life, $C_{x1} = C_{x2} = C_{xn}$, and if the surface concentration $C_s^T$ of these concretes are assumed to be the same value, it can be deducted that:

$$\left(\frac{X_1}{\sqrt{4D_1^T T}}\right) = \left(\frac{X_2}{\sqrt{4D_2^T T}}\right) = \left(\frac{X_n}{\sqrt{4D_n^T T}}\right) \tag{4}$$

In specification, only short-term chloride penetration profiles and short-term diffusion coefficients or other indicators of quality are likely to be available. For the same set of concretes, the chloride level at $x_1$, $x_2$ and $x_n$ at a shorter term time span $t$ can be determined in the same manner as their long-term chloride levels given above. In this case, the chloride level found will be significantly lower as illustrated in Fig. 1. Given the same range of similar concretes with the same short-term $C_s^t$, it can be found in the same manner as above that:

$$\left(\frac{x_1}{\sqrt{4D_1^t T}}\right)=\left(\frac{x_2}{\sqrt{4D_2^t T}}\right)=\left(\frac{x_n}{\sqrt{4D_n^t T}}\right) \quad (5)$$

It is intended that $x_1$, $x_2$ and $x_n$ correspond to the locations of $X_1$, $X_2$ and $X_n$ respectively. Hence, the long-term (e.g. at a given service life) chloride threshold level at these cover depths can be translated to another short-term chloride level at the same cover depths provided that (Eq. $\{[4]/[5]\}^2$):

$$\frac{D_1^t}{D_1^T}=\frac{D_2^t}{D_2^T}=\frac{D_n^t}{D_n^T} \quad (6)$$

i.e. the ratio of the short-term ($t$) to the long-term ($T$) diffusion coefficient of each concrete is constant. Maage *et al*. (2000) suggested a relationship described as follows:

$$D_n^t = D_n^T x\left(\frac{T}{t}\right)^{\alpha} \quad (7)$$

where $D_n^t$ is the 'achieved' diffusion coefficient calculated based on the chloride curve measured after a shot-term exposure time $t$; $D_n^T$ is the 'achieved' diffusion coefficient calculated based on the chloride curve measured after a design life exposure time $T$; and $a$ is a factor (alpha) which related the short- to long-term diffusion coefficient of the concrete.

Hence, Eq. (6) will hold if the sets of concrete considered have the same $a$. This alpha factor can be determined if there is a linear relationship between the diffusion coefficient and exposure time when both are plotted in a log scale similar to the one shown in Fig. 2. It is noted that α represents the rate of reduction of $D_a$ with the time of exposure.

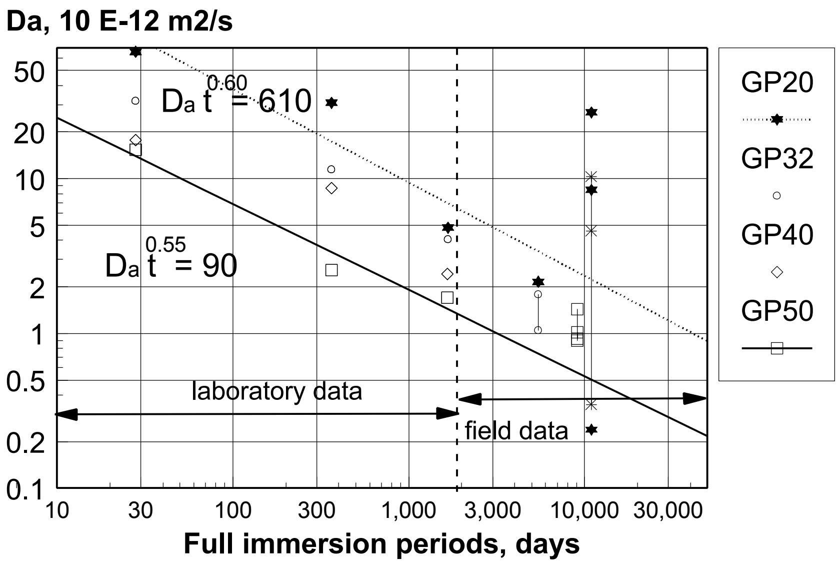


**Figure 2. Variation of $D_a$ with periods of immersion for GP concrete**

Hence, the concept of equivalent covercrete derived from short-term data is theoretically acceptable provided that:
the ratios of the short-term to long-term diffusion coefficients for the range of concretes considered are constant (constant α); and
the surface chloride concentration $C_s$ of the concretes are similar at the same time of t and T.

### 4.2 Limitation on the type of concrete

Let us explore when these two conditional requirements are met. From a theoretical viewpoint, the reduction of the apparent diffusion coefficient with exposure period is likely to be due to the maturity of concrete, i.e. the improvement in pore structure with hydration. Reasons other than maturity have been discussed earlier. Greater degrees or rates of reduction are expected in concrete with higher water-to-cement ratios and from cements with lower rates of hydration or pozzolanic reaction. Surface chloride concentration has been reported to be a function of surface porosity.

Experimental data and field data must also be examined to see where these conditions applied. In a major experimental program conducted at CSIRO, a range of structural concretes was proportioned from four commonly used cements and a water-reducing agent. These cements are designated as follows:

- GP – a general-purpose cement;
- FA or GP/30FA – a 30 wt% fly ash blend;
- SF or GP/7SF – a 7 wt% silica fume blend;
- HS or GP/65S – a 65 wt% slag blended cement; and
- GB blended cement concretes (FA, SF and HS).

Concrete of nominal strength grades of 20, 40 and 50 were manufactured in a laboratory to give a slump of 75 ± 25 mm. Standard 100 mm diameter cylinders were wet cured for 28 days for compressive strength determination. Other samples were subjected to 7 days of sealed curing in plastic sheet and were subsequently stored in standard laboratory conditions until the age of testing. Testing procedures used for a range of short-term properties have been previously described (Sirivivatnanon & Khatri 1997).

Concrete samples sealed cured for 7 days were subjected to full immersion in 3% NaCl solution at the age 28 days. Chloride penetration was determined by drilling and analysis of water-soluble chloride of samples collected at different depths from the exposed surface. Chloride profiles obtained after immersion periods of 28 days, and 1 and 5 years are used to study the effect of strength grade and cement type on the surface chloride concentration $C_s$ and apparent diffusion coefficient $D_a$, as shown in Figs 3 and 4. In Fig. 3, the value of $\alpha$ of various concretes is shown for a range of concrete grades. It should be noted that $\alpha$ represents the rate of reduction of $D_a$ with time of exposure. Both FA and HS showed higher values but rapid reduction of $\alpha$ with increased strength grade, especially for lower grades of concrete. This implies greater change in the degrees of hydration with time of FA concrete compared to GP concrete with similar strength grade. The same effect is observed in lower grades HS concrete, but the effect evens out from Grade 40 upwards. For FA, HS and GP concretes, the reduction in $\alpha$ with increasing grades indicates slower reduction of $D_a$ with increases in exposure time for these high grades concrete.

The SF concrete, on the other hand, behaves in the opposite manner, with increasing values of $\alpha$ with increased grades, i.e. the higher the grade of SF concrete the more rapid the reduction in $D_a$ with exposure time. In grade 32 and below, SF concrete showed very low $\alpha$ or a slow reduction of $D_a$ with exposure time. Silica fume is a very reactive pozzolan and hence is expected to react very rapidly with very little late reaction. This could explain the lower $\alpha$ for low-grade SF concrete, but could not predict the behaviour of higher grade SF concrete.

For grade 40 and 50, the common grades specified for aggressive marine environments, $\alpha$ is within ±0.07 for all four binders with the exception of grade 40 FA concrete. The $\alpha$ value can therefore be considered to be approximately constant for these concretes within this limiting range of grade 40 to 50. However, the variation of $\alpha$ in grade 32 and below is significant.

The variation of surface concentration of concrete with various cements was not significant, and the effect of cement was not distinguished in the analysis. The influence of strength grade and exposure time on $C_s$ is shown in Fig. 4 for a set of laboratory-immersed samples and field data collected from concrete piles at Iluka Wharf in northern New South Wales. It is clear that for a particular immersion period, the variation of $C_s$ between grade 40 and 50 is very small and thus can be considered a constant. This satisfies the second conditional requirement.

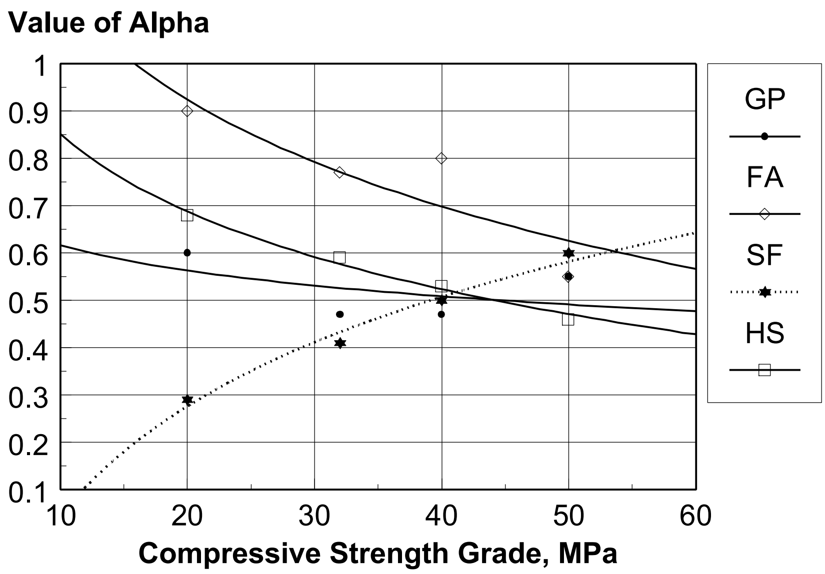


**Figure 3. Variation of a with compressive strength grade for concrete with different cements**

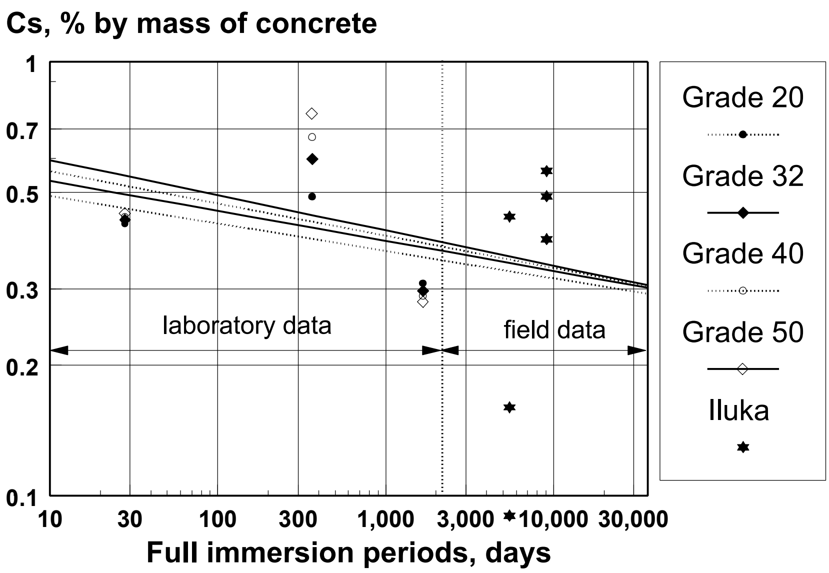


**Figure 4. Variation of $C_s$ with period of immersion**

Thus, the concept of equivalent covercrete is applicable, with a range of equivalent covercretes derivable from short-term chloride penetration data, for grade 40–50 concretes proportioned from a range of cements commonly used in Australia. In the case of FA concrete, the concept is perhaps more conservative than other cements, as α is higher than those obtained from GP, HS and SF concrete. Equivalent covercrete cannot be derived from short-term chloride profiles for concrete grade 32 and below because of the significant variation in α. However, if one confines to GP and HS concrete, the approximation holds for lower grade concrete as seen in Fig 3.

## 5 Specifying 'Quality' and 'Cover' versus 'Covercrete'

In order to specify a set of 'quality indicators' and 'cover' to achieve a consistent design life, the relative importance of each component must be examined in relation to a given design life. From Eq. (3), it can be deduced that the design life $T$ is proportional to the square of cover and inversely proportional to the diffusion coefficient:

$$T \propto Cover^2, \frac{1}{D_a}$$

In other words, the design life is far more sensitive to the variation in the cover depth than the cover quality. A lack or shortfall in cover depth has a multiple effect on the design life.

In most standards and specifications, the emphasis in specifying and achieving quality does exist and must be maintained. However, a great deal more effort must be given to specifying the correct cover depth and, most important of all, the means of controlling actual in-situ cover during construction. The need for and a method to control concrete cover have been discussed (Sirivivatnanon & Cao 1991).

## 6 Discussion

It has been shown that the resistance of concrete to chloride ion penetration can be characterised by the chloride profile or an apparent chloride diffusion coefficient ($D_a$) and a surface chloride concentration ($C_s$) of the profile. The profile and $D_a$ at time $T$ that corresponds to the design life are the ideal performance indicators. In practice, only short-term chloride profiles and $D_a$ at a more realistic time frame *(t)* are more likely to be available. It has been shown that the derivation of equivalent covercrete from short-term chloride profile is possible, provided that:

- ratios of the short-term to long-term diffusion coefficients for the range of concretes considered are constant (constant α); and
- the surface chloride concentration $C_s$ of the concretes are similar at the specific time t and T.

Examination of a large range of long-term Australian data showed that both conditions can be reasonably satisfied for concrete grade 40 and above. This applied particular to GP, HS and SF concrete where the rate of reduction of apparent diffusion coefficient $D_a$ with time, α, is similar. FA concrete has a slightly higher rate of reduction of $D_a$ with exposure time. Hence, more conservative results are expected from its inclusion in this concept of obtaining equivalent covercrete from short-term chloride profile. It is clear from the spread of α in lower grade concrete, shown in Fig. 3, that equivalent covercrete cannot be universally derived from short-term chloride penetration profiles for these concretes.

The use of chloride penetration profiles as performance indicators is considered more direct and hence superior than $D_a$. The concept of equivalent covercrete enables the full use of the chloride penetration profile. At any given cover, there is a chloride level that is critical to the performance of steel reinforcement at the corresponding location. Both $D_a$ and $C_s$ are necessary in defining the chloride level at a particular cover. It is much more difficult to consider both variables concurrently as a combined performance indicator. The use of $D_a$ on its own therefore implies the assumption that $C_s$ is constant.

It has been shown that the advantage of using 'equivalent covercretes' is the fact that they result in a unique design life, as derived from the chloride penetration profile. In practical terms, equivalent covercrete can give the designer more flexibility in using either a variety of covercrete in various components of a structure for a uniform service life, or one class of concrete and a range of cover depths to cater for different macro- and microclimatic (exposure) conditions. There is also more flexibility in reinforcing steel detailing to simplify construction. Practical limits, such as the classes of concrete available locally and the maximum cover limits to contain secondary cracking, will also set boundaries for both the *quality* and *quantity* of the concrete chosen for covercrete.

## 7 Conclusions

The concept of equivalent covercrete for concrete exposed to marine conditions or de-icing salts has been clearly defined. The fundamental principle is that all equivalent covercretes provide the same protection against chloride penetration to the steel reinforcement and hence the same design life. This can be achieved by evaluating and equating a chloride level at particular covers of a range of concretes after long-term exposure. By assuming that diffusion is the main transport mechanism for chloride ions, two conditions were found necessary for equivalent covercrete to be derived from short-term chloride penetration profiles. They are the use of a range of concretes with similar surface chloride concentration $C_s$ and the same rate of reduction of apparent chloride diffusion coefficient, $D_a$, with time $\alpha$. Both conditions are met for all the concretes of grade 40 and above, with more conservative equivalent covercrete expected for FA concrete because of the higher $\alpha$ value, especially for grade 40 FA concrete. The concept can form the basis for modern performance-based codes and specifications.

## 8 Acknowledgments

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# Probability Of A Dwelling Surviving Or Failing

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**Summary: Determining expected lifetimes and accompaning survival distributions for housing components was the focus for a project undertaken for the Queensland Department of Housing. The objective was to enable the estimation of the probability of a dwelling survival based on the current condition of the property. Initially a parametric solution to this problem based on the age of the property was employed. However, while this was adequate for a selection of properties, due to the heterogeneous nature of the portfolio it was inaccurate for individual properties and a more refined solution was required. The data source for the project was information collected by property inspectors for the 55,000 dwelling stock currently managed by the Department. In all 632 fields of data were collected on each dwelling.**

**This paper discusses the statistical analysis carried out to identify the survival distribution(s), both parametric and non-parametric, and their appropriateness for the problem at hand. Also discussed is the methodology employed to weight survival probabilities of the components to provide an estimate of the probability of survival for a single dwelling. The objective of the project was to understand the nature of the heterogeneity in the dwellings within the portfolio and ultimately improve the reliability of the predicted survival probability for each dwelling. The analysis determined that while a parametric distribution could have been utilised, the non-parametric method produced a more realistic and practical solution.**

**Keywords. Condition index; statistical analysis; survival distributions.**

## 1 INTRODUCTION

A property condition index has been developed for the Queensland Department of Housing, utilising the data collected in the Property Condition Appraisal Project (PCAP) on all their houses in Queensland, to provide managers with a consistent standardised measure of the current condition of a selection of properties within the property portfolio. A prototype software package has shown that the Property Standard Index (PSI) can be calculated for a single dwelling, any group of dwellings or all dwellings. However the interpretation of the index on individual property would require careful consideration by the users due to the generalisation of part of the index calculation and the heterogeneous nature of the property population.

The model developed used details on the current maintenance needs for a property and information on the standard of the property compared to the current standards required of the Queensland Department of Housing housing stock. The result was supplemented with a weighting scheme to allow for expected maintenance needs beyond the ten-year horizon. The forecasting algorithm utilised for the weighting was the Gamma function to provide a probability of failure for a property, based on its current age. The use of this model was determined through analysis of expected component failures and tested through simulation. The justification for its use was detailed in The Property Standard Index Final Report (Tucker et al, 2000). The algorithm used in the application is expressed in equation (1).

$$AF_i(t) = \frac{1}{\Gamma(a)b^a}\int_0^t t^{a-1}e^{-(age/b)}d(age) \qquad \textbf{Equation 1}$$

where $AF_i$ is the age function result for property $i$ at time t, and $a$ and $b$ are the shape and scale parameters of the Gamma function ($\Gamma$). A limitation though was that the property age was the differentiating variable within the algorithm and this required a belief that properties aged in a similar way. This is not true for all properties, thereby limiting the use of the model a single property level. Given the model was to facilitate decisions on selling, maintaining and reviewing the building stock of Housing Queensland, interpretation at a single property would be required regularly. To address the limitations of the current model a methodology was devised to measure accurately the weighting needed for the property Standard Index and incorporate the population variability.

## 2 METHODOLOGY

Statistical methods that fall under the heading of Survival Analysis were primarily developed in the medical and biological sciences, but they are also widely used in the social and economic sciences, as well as in engineering, reliability and failure time analysis. Essentially, the primary difference in the methods used in Survival Analysis and other fields is the need to handle censored or incomplete data. Data used in Survival Analysis generally contains a measure of the 'life' or time to event of a variable. Often the event is not observed and the researcher is limited to knowledge that the variable was 'alive' at some point but not of when or if failure occurred. This type of data is known as right censored data and is the most common form of incomplete data. Similarly the data could be left cemsored, interval censored or doubly censored, which presents the greatest challenge to the researcher. In this study the data contained examples of each of the censoring types which added to the complexity of the problem.

Modeling of survival times is based on two distinct approaches – parametric and nonparametric. The parametric methods involve fitting a distribution to the data and estimating suitable paramaters to the distribution enabling estimation of event times and survival probabilities. From the data a survival distribution is identified and using a least squares algorithm or maximum likelihood estimation, the parameters are estimated and tested for goodness of fit. Plots of the survival function, hazard, and probability density for the observed data and the respective theoretical distributions provide a graphical check of the goodness-of-fit of the proposed distribution. The major limiting factor for using parametric methods for this problem was the need to monitor the parameters regularly and make adjustments to incorporate variability due to improvements and other changes in property management techniques to maintain the accuracy of the estimates. Non-parametric methods are distribution free methods and rely on the quality of the data and homogeneity of the data to provide the estimates required. Some of the procedures employed here were life table computation and Kaplan-Meier survival function estimation.

The Kaplan-Meier Product-Limit Estimator, $\hat{s}(t)$ ,was developed by Kaplan and Meier (1958). The model is

$$\hat{s}(t) = \prod_{t_i<t} \frac{r(t_i) - d(t_i)}{r(t_i)} \qquad \textbf{Equation 2}$$

where $r$ is the number at risk and $d$ is the number of deaths at time $t_i$. The advantage of the Kaplan-Meier Product-Limit method over the life table method for analysing survival and failure time data is that the resulting estimates do not depend on the grouping of the data into a certain number of time intervals although for our purpose time has been divided into discrete yearly intervals.

The procedures outlined are essentially methods for estimating the distribution of survival times from a sample. In practice, for predictive purposes, identification of the survival function in the population is generally the target strived for and there is a need to make inferences about the results. In this case the entire population was utilised thus the main concern was how the distributions will evolve over time.

One of the difficulties to overcome was that the hazard functions may be influenced by extreme events. Ensuring sufficient observations limits this effect and if this is not possible a measure of the accuracy of the estimate will provide the user of the potential for error in the estimate.

The results of the analysis of the important components were also combined with prior knowledge using the Bayesian paradigm to test the suitability of the output as a measure of survival probability. This essentially tested whether there was any significant difference in the results compared with the perceived life of a component.

## 3 ASSUMPTIONS

For this analysis, minimal assumptions were made. Firstly, an assumption of independence was made. It was assumed that failure of a component did not affect any other component and while not strictly correct, especially with regard to failure of paint and the effect on the protected component, the assumption that components are (weakly) independent ensured model development excluded the need to incorporate conditional probabilities. This assumption could be tested by simulation at a later time.

The expected failure time of components was assumed to be the time until the median failure time for the component population allowing for censored observations. For implementation, or at any future point, this could be modified to ensure the application is using the best information available. The estimate of the component life used will make a significant difference in the final weighting for the dwelling because certain major components, such as roofing, require replacement at varying intervals. A reduction of 10% in component life may mean that a dwellings survival probability is in error and will give a more pessimistic perception than may actually be the case.

The data included information from all identified repairs including some failures which may be due to obsolescence other than physical decay. This required assumptions to be made about the reason for scheduled maintenance. Some of the likely reasons for obsolescence speculated include:

- Health and Safety – the component requires replacement to eliminate the risk of injury
- Physical – the component has deteriorated and the physical collapse is imminent
- Economic – no longer economic to maintain
- Functional – the component no longer functions for the purpose it was originally intended
- Technological – advances in technology had rendered the component outdated and replacement is demanded as a result
- Social – the component is no longer socially accepted
- Legal – the component is legally required to be replaced

It was determined that any of these reasons were fair and should be catered for. Another cause of maintenance is unreasonable use; this is an unacceptable cause of maintenance but has not been provided for in the analysis as no distinction could be made in the data.

Also, expected life will be affected by the frequency of maintenance visits and component finishes. A number of similar components and material types were grouped together. Mapping of the components into the categories was made using two criteria, mathematical and conceptual. The first was on the basis of similar failure patterns, therefore mathematically sensible, and the second was that the classifications had to be conceptually correct, with only related components grouped. This reasoning was to ensure the categories were both statistically correct and acceptable in the eyes of the portfolio managers. Every effort made to ensure appropriate grouping and the groupings were reviewed by DoH confirming the groupings were reasonable. Use of the application over time may lead to identification of incorrectly grouped classifications and a need to redefine the categories. This would not be a difficult process and ongoing assessment of the classifications is recommended.

For the purpose of this analysis, the factors accepted as influential or differential factors were those identified and reported in Property Condition Index: Final Report (Tucker et al, 2000). Covariates other than the dwelling type and component materials were not assessed here. The provision for the co-variates documented lead to a significant reduction in variation for the components. Further improvements could be gained through recognition of systematic differences in the survival rate as compared with that expected after the computer program for the Property Standard Index has been operating.

## 4 PLAUSIBLE MODELS

The objective was to predict the expected survival probability for a property with an acceptable level of accuracy. To meet this objective a range of models were hypothesised and tested with the preferred model being the one that best addressed the following criteria. These criteria are listed below in order of importance:

~~a)~~The model that best predicts the perceived likelihood of failure.

~~b)~~The model should minimise the variation by capturing the dynamic and systematic effects leaving only minimal random effects.

~~c)~~The model should have minimal impact on the computer application in terms of ongoing management and maintenance.

~~d)~~The model should be computationally efficient.

Consideration of the data to be used by the computer application once implemented and an awareness of the implications of the methodology so far as computational efficiency was concerned ensured that the final model could be dynamic enough to capture the change in the stock profile over time and minimise the calculation time needed to compute the Property Standard Index rating. These considerations instigated a change in strategy on a number of occasions.

The estimation of the expected survival probability for a property required development of a methodology that would identify and then systematically evaluate each component of a dwelling, calculate a survival probability for the component and generate a weighting coefficient dependent on the other components contained in the property. The resulting value must be constrained within the bounds of zero and one and be representative of the dwellings expected useful life after consideration of the programmed maintenance. Fundamentally, the estimate will be a probability of survival for the dwelling given identified maintenance is carried out as planned, the goal being that the resulting PSI rating will better reflect the perceived rating as determined by the housing inspectors. The final methodology for the ‘age function’ should complement the other elements of the PSI algorithm by retaining the simplistic nature and remaining objective.

The general model would be:

$$\Pr(Survival_i) = \sum_{c=1}^{C} l_c p_c \qquad \textbf{Equation 3}$$

with $l_c$ representing the component weighting and $p_c$ a measure of the probability of survival for each component ($c$) of C components in property $i$.

A range of models were hypothesised as possible soultions. The model alternatives were:

**Model 1.** Parametric survival model utilising the 'important' components based on the Pareto principle. The approach here was to determine through lifecycle cost analysis and the Pareto principle (80% of the costs are due to 20% of the components) which components were most important in the life of a property and then identifying parametric survival distributions for these components. The component weighting could then be established and an estimate for the probability of survival for the property derived. A limitation of this option would be the need to monitor the estimates of the model parameters over time to ensure they remain accurate.

**Model 2.** Non-parametric models utilising the 'important' components. This model utilised the information gained pertaining to important components and fits probability of survival based on the historical occurrences only. This model would not require parameter estimation.

**Model 3.** Parametric survival model utilising all components. The goal of this analysis was to better understand the data and draw out, if possible, a single parametric survival distribution or if needed, a range of distributions to fit the data. Identification of the parametric distribution(s) was the major objective, as poor fitting distributions would limit the precision of the model. Once identified, a model was then developed with parameters adjusted to fit each component. A methodology to aggregate the survival probabilities for the components to a single dwelling had to be formulated to produce a result for the property.

**Model 4.** Non-parametric model utilising all components. Model 4 utilises the data to describe the likelihood of failure and convert this to a survival probability. As was the case for Model 2, this empirical method relies heavily on the historical data but, assuming the reliability of the data, should provide dependable results. The use of empirical probabilities has wide spread application in survival analysis, particularly in biological and the medical fields (Lee, 1992; Collett, 1994).

## 5 DATA

The available data used for the analysis also impacted on the analysis process. Initially the analysis was to utilise the historical maintenance database, which entailed restructuring the data to enable statistical analysis to be carried out. The database consisted of over 3,000,000 maintenance records for the 55,000 dwelling units currently managed under the portfolio of Housing Queensland. This data had to be modified to correspond with the data needed for the PSI application. Ultimately this proved to be unsuitable for the problem and the PCAP data was finally used but the maintenance data will serve as a source of confirmation at a later stage. The use of this data also had some of its own difficulties. The preliminary analysis was carried out using the attribute/sub attribute numbers to identify components used in the PCAP data and the prototype PSI application. The final PSI application relied on using class and characteristic numbers as identifiers and the transition to class and characteristic numbers required the attribute/sub attribute to be mapped to the new codes before the analysis process could resume. This proved to be a challenging and time consuming task.

The preliminary analysis of the data sought to establish which components were most important when estimating the probability of survival of a dwelling. The approach used here was to value the component, estimate the number of replacements expected during the dwellings life (based on expert advice) and order the components by the total cost. This principally is a simplified version of lifecycle analysis where the only lifecycle cost considered is the replacement cost. The purpose was to enable the survival analysis to focus on the elements that offer the greatest cost burden throughout the life of a property, but also provided an estimate of the relative importance of all of the housing components. After generating the expected costs, the Pareto principle, the principle that 80% of the costs could be generated by 20% of the components, was the rule used to determine the most important components.

Initially the data used was restricted to data classified as right-censored lifetime data, which is the most common case of censoring, and the other data was used to test the distributions fitted. A subset of the data was set aside to be used for confirmation of the fit once the models were finalised.

## 6 SURVIVAL ANALYSIS

The key objectives for this analysis was to give an insight into the expected failure time of a component given the specific agents that are a possible cause of variation and either

1. Fit an appropriate parametric distribution to the observed distribution or
2. Develop a method of utilising non-parametric measures of survival probability.

After the elementary analysis the age distributions for both the failed and total populations of the different components and component types were plotted to provide an enhanced view of the distributions. The charts showed graphically how the different components and material types were distributed and the peak usage periods showed up well. The charts showed that it was unlikely a single distribution could adequately describe the various material types and fitting distributions to each type could also be difficult.

Following on from the descriptive analysis, a survival analysis was completed. Here an attempt was made to identify a survival distribution for the data. The first procedure attempted to fit a conventional distribution to the data; in particular the Weibull, Extreme, Normal, Lognormal, Logistic and Log-Logistic distributions were compared. The analysis indicated that the log logistic distribution may be a suitable distribution to describe the failure of roofs. The tails of the distribution were of some concern and it is in these regions where more precision was required to improve on the existing model. Also when the distributions were closely assessed there were systematic differences in the estimations with survival probabilities for metal roof types generally over estimated and tiled products under estimated.

Figure 1 displays the distributions of the roof types that have failed, in essescence the chart shows the likelihood that a failed roof failed at each specific age. Assessment of roof types suggested that unpainted corregated galvanised zinc roofs and terracotta tiled roofs both have distributions that could be modelled successfully but the other types seem to be bimodal in each of their distributions and would be difficult to model using parametric techniques. The cause of the multiple peaks, evident in Figure 1 as significant decreases in the survival probability, is believe to be failure of re-roofed properties but this has to be confirmed.

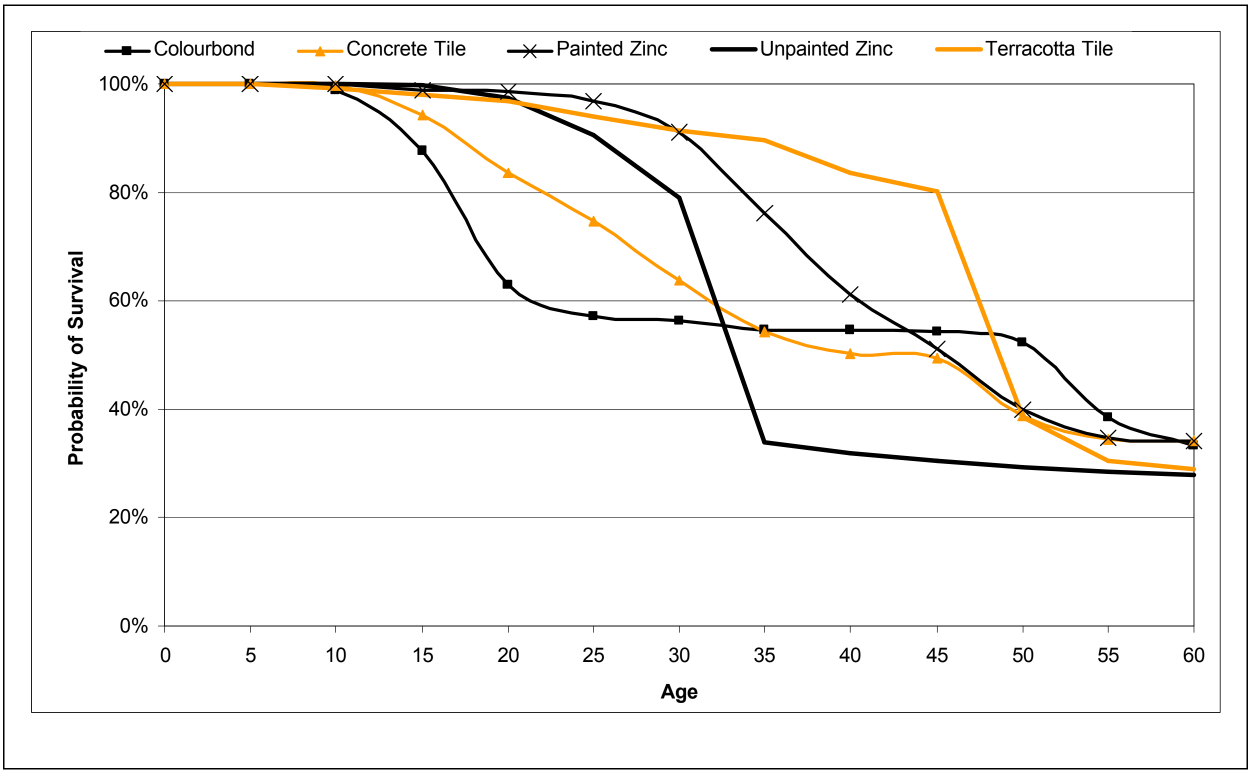


**Figure 1 Empirical survival distributions for selected roof types**

The goodness of fit for the distributions was assessed using the Chi-square statistic. The method required the computation of the likelihood of the data given the estimated parameters for the distribution selected and comparison with the life table probabilities. For most components, the statistic was not statistically significant at the 95% level and on the occasions where it was marginal; the graphical output indicated the area of concern was the distribution tails. Effort was put into finding some common factor between the components displaying the unusual properties but no commonality was determined. As a result of the analysis, either distributions such as the Beta or Gamma would need to be used with the parameters adjusted to fit the data, which would require a heavy and regimented maintenance strategy for the distribution parameters or the use of the parametric distributions would require the distribution tails to be modified. Again it seemed the objective would not be met unless the heterogeneity could be better captured and this would not be accomplished with a single distribution.

Another possibility was to introduce a ‘change point’ where new parameters were introduced for the tails of the distributions. This would have require a general distribution for the components that failed as expected, a joint distribution for those that failed around the ‘change point’ and a modified distribution for the components that exhibited the unusual properties. While this is not unachievable, it is impracticable, going against criteria c in our preferred model criteria. Given that the variability could not be adequately described in a consistent manner it was decided to test non-parametric methods.

The Kaplan-Meier method, as described earlier provided the methodology needed. This method enabled the probability of survival to be determined for each time point using the whole population. For conciseness, the components were classified into a minimal number of categories to limit the imposition on the computation for the application. Essentially a count of the number of surviving units for each age is completed at the end of the period and this is divided by the count at the beginning. This resultant hazard rate is multiplied by the previous periods survival probability and a survival probability for that period is determined.

This methodology provides a relatively simple and practical solution and, providing the categorising of the components is correct, means the application will not require on-going management for the distribution parameters as was anticipated. Further to this, in time as the database is more populated the accuracy of the predictions will improve with out any effort in managing the model.

Using the Kaplan-Meier method to calculate estimates of the probability of survival provided the basis for a suitable model for the PSI application. Further, from the survival probability tables it is possible to calculate the hazard rate for a component at a point in time to provide an estimate of how many of a specific component might fail in the given period. Where insufficient data exists a default table will be referred to to provide the information needed. As mentioned the reliance on this table will reduce in time.

Once the methodology to determine the survival probabilities had been defined, the focus moved to development of a methodology to weight the components correctly for aggregation to dwelling level.

## 7 COMPONENT WEIGHTING

The component weighting is needed to provide a proportional weighting to each component as it is represented in the specific dwelling. Thus, the methodology had to incorporate both the differences in the dwellings and the dynamic nature of the portfolio. While there are many dwellings that are homogeneous, the range of component types that make up the dwelling could differ and accordingly the probability of survival should differ. In all, there are two hundred components in the appraisal forms that expand to the 1718 possible component-types, so the likelihood is that any two dwellings could have at least one difference. More importantly, there is a reasonable chance that two dwellings could differ significantly.

Also, as was evident in the analysis, the portfolio is dynamic, changing with the introduction of improved, more suitable products and with the needs of the population. The methodology needed to incorporate the portfolio dynamics and component differences. To meet this challenge a process of establishing component weights was developed that scanned the database for the specific components installed at each dwelling, then searched for the modal cost for replacement of the component as estimated by the housing inspectors as it was believed that this would be a suitable and consistent estimate of the true maintenance costs. Once returned, the modal costs were summed and the cost for each component was then divided by the total component costs for the dwelling providing a standardised weight for each component. Thus, each component received a weighting proportional to it cost for the dwelling and each dwelling was assessed only for its components. This process is expressed in the equation below.

$$l_c = \frac{m_c}{\sum_{c=1}^{C} m_c} \qquad \textbf{Equation 4}$$

where $l_c$ is the component weighting and $m_c$ is the modal cost for each component, $c$, *of C components*. The costs are dynamically extracted from the PSI database and for those where there are insufficient occurrences to determine a cost, the costs are obtained from a default table currently being used to supplement the data for the estimation of missing standards. Again the use of this table will reduce as the database is populated in time. Also, this process, while heavy in computation time at each upload, minimises the application development required and minimises the ongoing maintenance for the computer application.

Implementation of the final model requires a table to be defined with the survival probabilities, expected to be approximately ninety classifications by 60 (years) which will be populated using the algorithm from the PSI database. A further column will be added to the house attributes table to include the component weighting. The expectation is that once calculated, the component weighting is unlikely to alter and only those dwellings with significant alterations will require recalculation. The survival probabilities will require recalculating with each new upload.

## 8 NEW MODEL

Based on the analysis carried out the new ‘age function’ to replace Equation 1 in the Property Standard Index algorithm is:

$$AF_i = \sum_{c=1}^{C} l_c p_c \qquad \textbf{Equation 5}$$

where $l_c$ is the component weighting based on its relative cost as described in equation (4) and $p_c$ is the probability of failure of component $c$ of C components. Calculation of the survival probability for a dwelling $i$ will require the application to determine the age of the dwellings components, select the survival of each component from the table and multiply this by the component weighting. These values are then summed to provide an estimation of the dwellings survival probability. Figure 2 displays a comparison of the difference in probabilities between the two methodologies, in the case displayed the property was a brick clad property with a terracotta tiled roof. The obvious benefit of the new methodology compared with the earlier model is in the rise in survival probabilities for a property once major maintenance has been completed, such as kitchen refurbishment and internal painting at seventeen years and again at 35 and 53 years. With other properties that require more significant external painting or other significant components the decline and increase is more pronounced. In this output the maintenance was carried out as the component failed with no consideration given to programmed maintenance. The new methodology provided a reasonable approximation to the initial methodology falling within the confidence bounds for a single property at most time periods.

Go Back

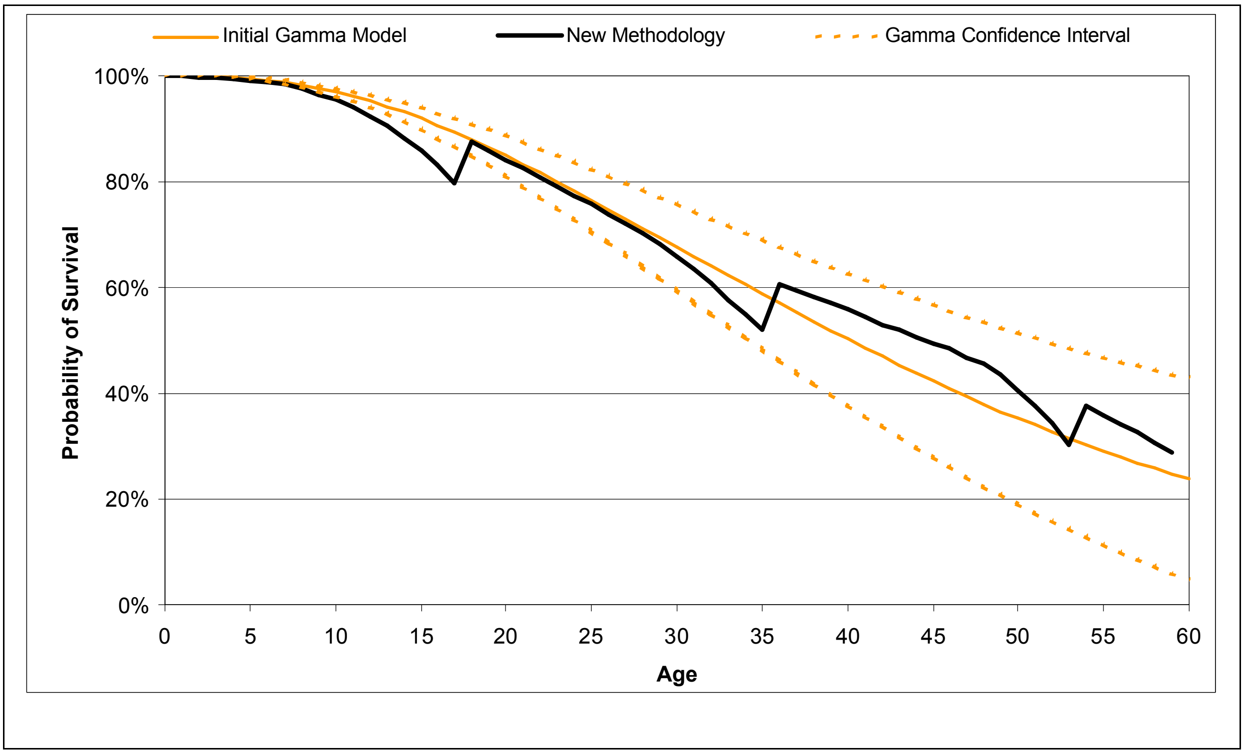


**Figure 2Comparison of initial and new methodologies**

In the first instance, determination of the age of a component is assessed by checking for the inspector's estimate of years to repair date. Should the value be between 10 and 3 exclusive then the age can be calculated as the median failure age less the years to repair. Should the inspectors estimate be greater that 9 then the last date of repair within the application database will be checked. Should this be null then the estimation of the age will be defined as the minimum of the median failure time less ten and the calculation of the modulus of the property age given median failure time of the component.

$$\min(\bar{s}-10, |dwelling(age), \bar{s}|) \qquad \textbf{Equation 6}$$

Generally, the impact of the age estimation is likely to be minimal on the overall survival probability for the dwelling. However there will be some instances where the impact will be noticeable. The onus will be on the users within the department to interpret the output bearing in mind the infancy of the database which will, in time, be more complete and provide the users with more precise output.

## 9 CONCLUSIONS

The project set out to develop a practical solution to a significant problem of estimating the probability of a dwelling survival based on the current condition of the property. The model developed has met the criteria set out for a suitable model and should prove to have provided the solution needed. The new model will be incorporated in to the Property Standard Index computer application which is a software package developed for the Queensland Department of Housing to provide managers with a consistent standardised measure of the current condition of a single dwelling, any group of dwellings or all dwellings within the property portfolio.

The recommended model captures all component failures regardless of the reason for failure, whether physical deterioration, lack of functionality, or economical obsolescence. It is based on a belief that all information should be utilised. However the methodology allows for the exclusion of failure causes if desired, providing the cause can be specifically identified.

The final model recognises that components in dwellings do not all fail simultaneously and uses the varying installation times of housing components to estimate the survival probabilities of the components, then by weighting the components proportionately, the estimates can be aggregated to dwelling level providing a more accurate estimate of the survival probability for the dwelling than the original methodology. This new methodology will allow portfolio managers to select a single property from the Property Standard Index application and be confident in the accuracy of rating calculated.

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# Prediction Of Chloride Penetration Into Concrete Exposed To Various Exposure Environments

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**Summary**: **This paper presents the results from a study of prediction model for chloride penetration into concrete exposed to various exposure environments including alternative wet-and-dry environment. A few years ago, a scientific model called ClinConc was developed from our previous work. The model is essentially based on the current knowledge of physical and chemical processes involved in the chloride transport and binding in concrete and has been verified by using the field data from one to five years exposure under seawater. In this study, the model is further developed for the application to alternative wet-and-dry environment, such as splash zone and road environment.**

**The actual chloride profiles measured from the field exposure stations are used to verify the modified model. The predicted results are in general fairly well in agreement with the field data, especially the shapes of chloride profiles from alternative wet-and-dry environments. The limitations and needs for further improvement of the latest version of the model are discussed.**

**Keywords. Chlorides, concrete, modelling, prediction.**

## 1 INTRODUCTION

The numerical model ClinConc (***Cl in Conc***rete) for prediction of chloride penetration into concrete was first presented in the middle of 1990's (Tang & Nilsson 1994; Tang 1995). The model consists of two main procedures:

1. Simulation of free chloride penetration through the pore solution in concrete using a genuine flux equation based on the principle of Fick's law with the free chloride concentration as the driving potential, and
2. Calculation of the distribution of the total chloride content in concrete using the mass balance equation combined with non-linear chloride binding.

Not like other models, a unique character of the model ClinConc is that the chloride diffusivity, which can be determined by, e.g. the Nordtest method NT BUILD 492 (Nordtest 1999), is considered as a material property. It changes only when concrete is young, like many other material properties, such as porosity and strength. After an age of a half of year, this diffusivity becomes more or less constant according to the experiments (Tang & Nilsson 1992; Tang 1996). Another unique character of the model ClinConc is that the climatic parameters, such as chloride concentration and temperature, are used in both the flux and the mass balance equations. Therefore, the model can well describe the effects of exposure conditions on chloride penetration.

The original version of ClinConc was developed based on the field data up to two years exposure under seawater. Due to the difficulties in combining moisture transport, the application of the original ClinConc was limited to submerged zone only. When five-years field exposure data were available (Andersen et al 1998), it was found that the original ClinConc underestimated the chloride content in the zone closer to the exposure surface, even though it predicted the penetration depth fairly well. In other words, the surface chloride content tends to increase with exposure time even under submerged conditions. This increased chloride content cannot be explained by drying-and-wetting effect, like in the splash zone. Time-dependent chloride binding might be a potential reason, since the chloride binding isotherms used in the original ClinConc were those obtained in the laboratory after about two weeks equilibrium (Tang & Nillson 1993). The effect of alkalinity on chloride binding was also based on a limited investigation (Sandberg & Larsson 1993). In reality, the pore solution compositions may change due to leaching and penetration of different substances, resulting in different characteristics of chloride binding. Another possible reason is an increased saturation degree of the air voids near the surface. The saturation degree of the air voids will increase after such a long period of immersion, especially in contact with a salt solution. It is difficult, however, to model the saturation degree of the air voids. Therefore, the time-dependent chloride binding was assumed as a dominant reason for the increased chloride contents in the surface zone (Tang & Nillson 2000). After this modification, the agreement between modelled and measured chloride profiles becomes better (Tang & Nillson 2000b).

Very recently, the model ClinConc was modified again in order to make it applicable to various exposure environments including alternative wet-and-dry ones. In fact, nothing except for the exposure conditions has been modified in the latest modification. This paper present this latest modification and the verification of the model using chloride profiles measured from the fields under various exposure environments.

## 2 MODELLING FOR VARIOUS EXPOSURE ENVIRONMENTS

### 2.1 Exposure Conditions for Marine Environment under Submerged Zone

Under submerged zone, concrete is constantly in contact with seawater. This might be the easiest case when compared with other exposure environments. However, since both temperature and chloride concentration in seawater change with time, even in this easiest case it is still difficult to use constant boundary conditions for chloride transport in concrete. In the previous versions, both the exposure temperature and chloride concentration were assumed as a sine function. The sine function of annual average temperature has been well verified, but not the chloride concentration in seawater. Therefore, it was suggested in the latest version that average chloride concentration in seawater should be used unless the actual function of chloride concentration is known. An example of the exposure conditions for submerged zone is shown in Fig. 1.

### 2.2 Exposure Conditions for Marine Environment above Seawater

In the marine environment above seawater, such as splash zone or atmospheric zone, concrete is subjected to alternative wetting-and-drying. The wetting includes both salt water and rain. Owing to the complicated mechanisms involved in both the moisture and chloride transport, it is not an easy task to combine both moisture and chloride transports into a single model, even though some attempts have been done (Nilsson 2000; Francy et al 1996). On the other hand, it could be reasonable to assume that, under such a wet-and-dry environment, the chloride concentration in contact with the concrete surface alters between zero and a specified level. The wick effect due to drying is compensated by the effect of capillary suction due to re-wetting. Therefore, the chloride transport could be assumed dominated by diffusion in a saturated pore system, despite of wetting-and-drying processes. In this way, the difficulties in modelling of moisture transport could be skipped and the question becomes how to

| Chloride Concentration | | | Temperature | | |
|---|---|---|---|---|---|
| Annual minimum | 14 | gCl/l | Annual minimum | 2 | °C |
| Annual maximum | 14 | gCl/l | Annual maximum | 20 | °C |
| Annual Cl period | 12 | months | Frequency | 1 | cycles/year |
| Max. in the month | | (number) | First mean at | 120 | days of year |

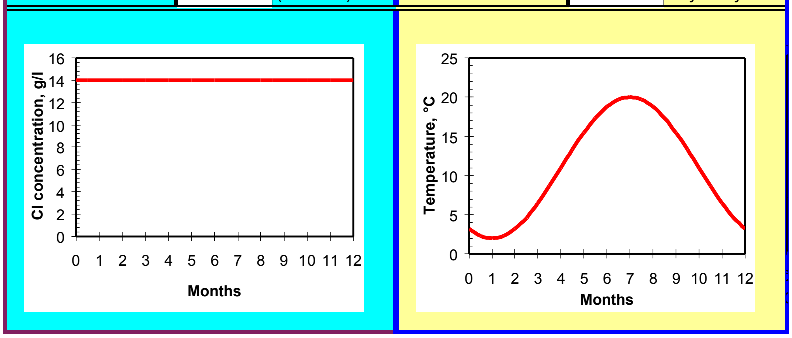


**Figure 1. Example of exposure conditions for submerged zone (Swedish west coast).**

define the chloride concentration curve. In the latest modification, a statistic normal distribution function was proposed to describe the annual chloride concentration, that is,

$$c_{0s} = \overline{c_0} \exp\left( -\frac{\tau^2}{2\sigma^2} \right) \tag{1}$$

where $c_{0s}$ is the chloride concentration in contact with the concrete surface, $\overline{c_0}$ is the average annual chloride concentration in seawater, $\sigma$ is the standard deviation that will be explained later, and $\tau$ is the time difference, which is a periodic function and expressed as

$$\tau = f(t) = f(t+nL) = \begin{cases} \left| \dfrac{t-t_{\mathrm{m}}}{L} - n \right| & nL \le |t-t_{\mathrm{m}}| \le \dfrac{(2n+1)}{2}L \\[2ex] \left| \dfrac{t-t_{\mathrm{m}}}{L} - (n+1) \right| & \dfrac{(2n+1)}{2}L < |t-t_{\mathrm{m}}| \le (n+1)L \end{cases} \tag{2}$$

$n = 0, 1, 2, 3\ldots$

where $t$ is the actual time, $L$ is the period, $t_{\mathrm{m}}$ is the time when the chloride concentration reaches maximum during the period, and $n$ is the integral of $t/L$.

The standard deviation $\sigma$ is a decisive parameter to the width of a statistic normal distribution curve and can be expressed as

$$\sigma = \sigma_{\tau} \frac{L_{\mathrm{Cl}}}{L} \tag{3}$$

where $L_{\mathrm{Cl}}$ is the chloride duration during the period ($L_{\mathrm{Cl}} \le L$), and $\sigma_{\tau}$ (= 0.15) is the standard deviation of $\tau$. It should be noticed that the time difference $\tau$ is a dimensionless parameter, implying that $t$, $L$, $t_{\mathrm{m}}$, and $L_{\mathrm{Cl}}$ must have the consistent dimension, which could be hours, days or months. Since the actual repetition of chloride concentration in splash zone is unknown, $L$ was simply assumed to be 12 months, that is, annually repeated in order to simplify the calculation. In this case the sine function of temperature is inapplicable, thus an average annual temperature should be used. Some examples of exposure conditions for the marine environment above seawater are given in Figs. 2 and 3.

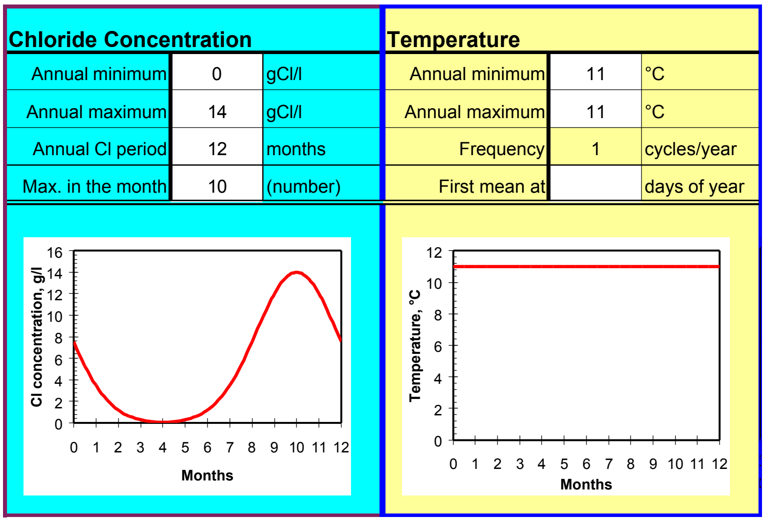


**Figure 2. Example of exposure conditions for splash zone (0~30 cm above seawater in Swedish west coast).**

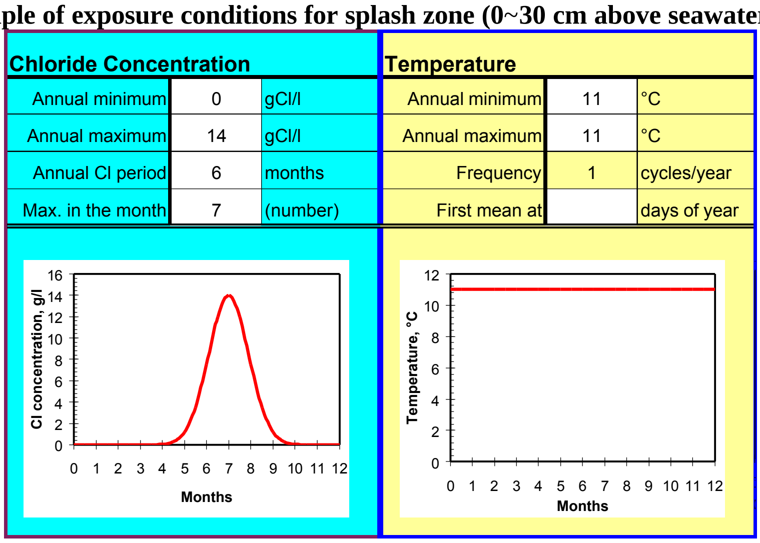


**Figure 3. Example of exposure conditions for atmospheric zone (30~60 cm above seawater in Swedish west coast).**

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### 2.3 Exposure Conditions for Road Environment Using De-Icing Salt

The same principles as described above can be used for the road environment. Thus the chloride concentration in contact with concrete surface is

$$c_{0s} = c_{\max} \exp\left( -\frac{\tau^2}{2\sigma^2} \right) \qquad (4)$$

where $c_{max}$ is the maximum chloride concentration during the period. The difference from the marine environment is that the chloride period (application of de-icing salt) is a more or less known parameter under the road environment, for instance, from November to March in the winter. Thus the sine function of temperature can be applied as in reality. However, the maximum chloride concentration $c_{max}$ in this case becomes unknown. From the field data obtained from the two-winters exposure along the highway Rv 40 between Borås and Göteborg it was found that, when assuming a maximum concentration of 50 g Cl per litre, the predicted profiles correspond fairly well with the field data, which will be presented later. An example of exposure conditions for the road environment is shown in Fig. 4.

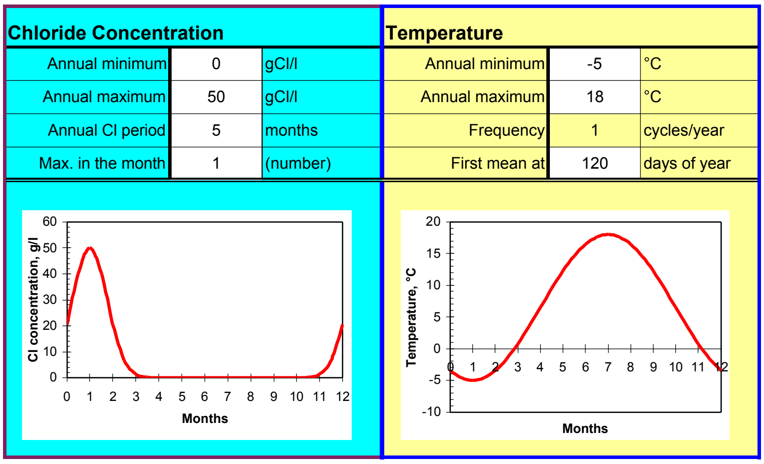


**Figure 4.Example of exposure conditions for a road environment (Highway Rv 40 between Borås & Göteborg, Sweden)**

## 3 VERIFICATIONS OF THE LATEST MODIFIED MODEL

From two Swedish national projects, many chloride profiles obtained after five years exposure under the marine environment and after two winters exposure under the road environment are available (Andersen et al 1998; Lindvall et al 2000). The chloride profiles from two types of binder and two water-binder ratios, which are commonly used in Sweden for infrastructures, were utilised to verify the latest modified ClinConc. The mixture proportions of concrete and relevant properties are given in Table 1, and the common parameters used in the calculation are listed in Table 2. The exposure conditions are as shown in Figs. 1 to 4. The results are shown in Figs. 5-8. Considering the very complicated mechanisms of chloride transport in concrete, we can conclude that the predicted results are in general fairly well in agreement with the field data, especially the shapes of chloride profiles from alternative wet-and-dry environments. This implies that the assumptions made in the latest modification for various exposure environments are reasonable and close to the reality.

**Table 1. Mixture proportions and diffusivity of concrete used for verification**

| Binder type | Water-binder ratio | Cement content $kg/m^3$ | Aggregate $kg/m^3$ | Air content $kg/m^3$ | Diffusivity* $D_{CTH}$ $m^2/s$ |
|---|---|---|---|---|---|
| SRPC | 0.40 | 420 | 1692 | 6.0 | $8.1\times10^{-12}$ |
| SRPC | 0.50 | 370 | 1689 | 6.4 | $19.9\times10^{-12}$ |
| 95%SRPC + 5%CSF | 0.40 | 420 | 1685 | 5.9 | $2.7\times10^{-12}$ |
| 95%SRPC + 5%CSF | 0.50 | 370 | 1683 | 6.0 | $13.4\times10^{-12}$ |

* Determined by the CTH method (NT BUILD 492) at an age of about 180 days.

**Table 2. Common parameters used in the calculation**

| Chloride Binding | Variable Diffusivity |
|---|---|
| Isotherm slope | Activation energy |
| $f_b = 3.57$ | $E_b = 40000$ J/mol |
| Non-linear exponent | Age dependent |
| $B = 0.38$ | $\beta_t = 0.152\ (w/b)^{-0.6}$ |
| Activation energy | $t_{D_a} = 180$ days |
| $E_b = 40000$ J/mol | Depth dependent |
| Time-dependent factor | None (steel form) |
| $f_t = 0.36\ln(t_{Cl} + 0.5) + 1$, | |
| where $t_{Cl}$ is the local chloride contamination time in years. | |

## 4 LIMITATIONS AND NEEDS FOR FURTHER IMPROVEMENT

Although the verification results show a good agreement with the field data, there still exist the following limitations:

- *Limited concrete type* In the above verification, the concrete type is limited to two water-binder ratios (0.4 and 0.5) and two types of binder (SRPC and 5% silica fume). Although these two types of concrete are very commonly used in Sweden for infrastructures, more types of concrete, especially HPC with low water-binder ratios and different types of binder, such as fly ash, slag, etc., should be used for verification.
- *Limited exposure time* So far the available data from the field exposure stations are limited to 5 years for marine environment and 2 years for road environment. This exposure time is relatively short when compared with the whole service life of concrete structures. More data from the long term exposure fields, especially with traceable exposure environments, are needed for a better verification.
- *Characterising exposure environment* In the latest modifications of the model, the alternative wet-and-dry environment is described using statistic normal distribution functions. The question is how to determine the key parameters $L_{Cl}$ – chloride duration for marine environment in equation (3) and $c_{max}$ – maximum chloride concentration for road environment in equation (4). Some simple methods are needed for characterising different exposure environments.

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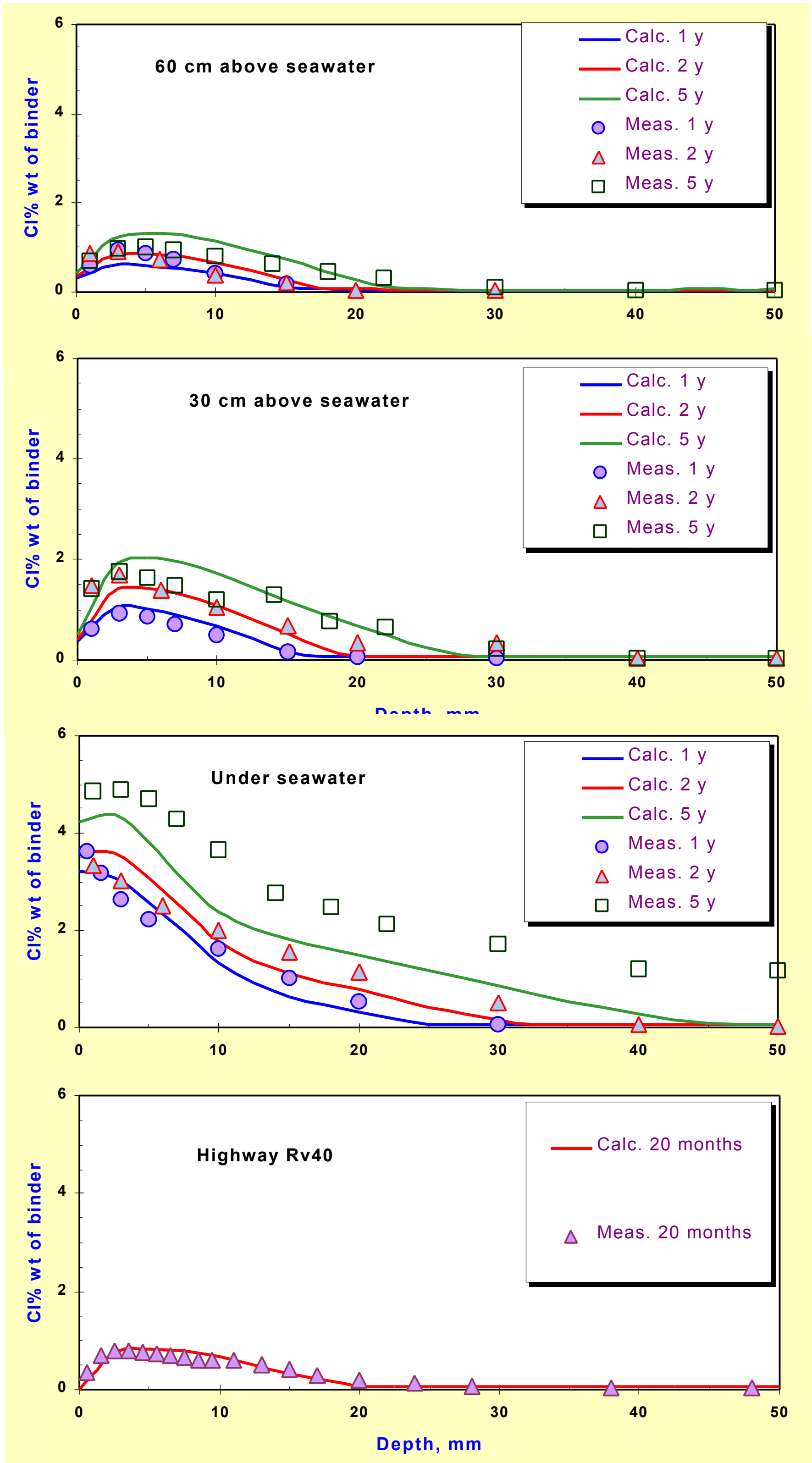


**Figure 5. Example of the predicted chloride profiles. SRPC w/b 0.40.**

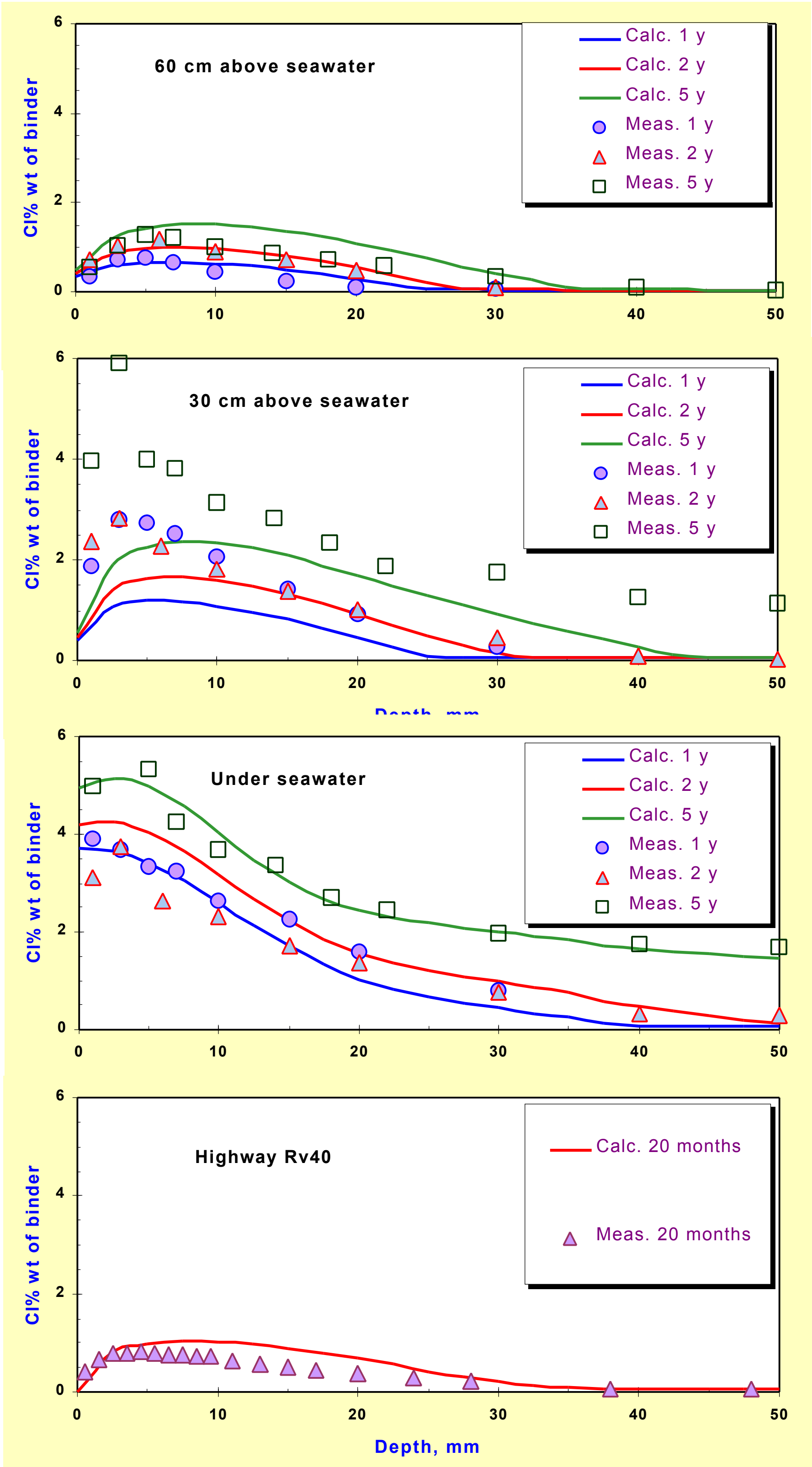


**Figure 6. Example of the predicted chloride profiles. SRPC, w/b 0.50.**

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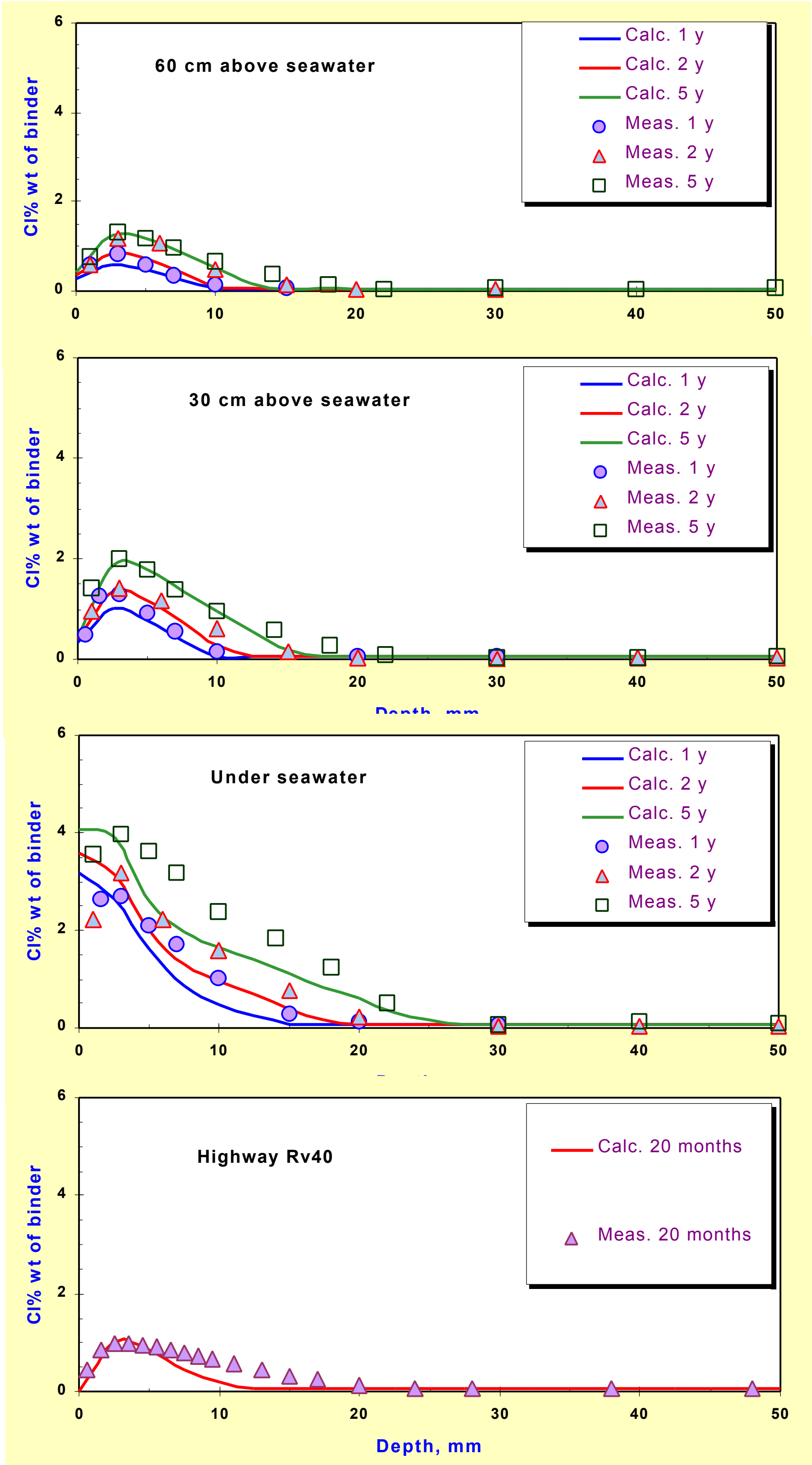


**Figure 7. Example of the predicted chloride profiles. SRPC + 5%CSF, w/b 0.40.**

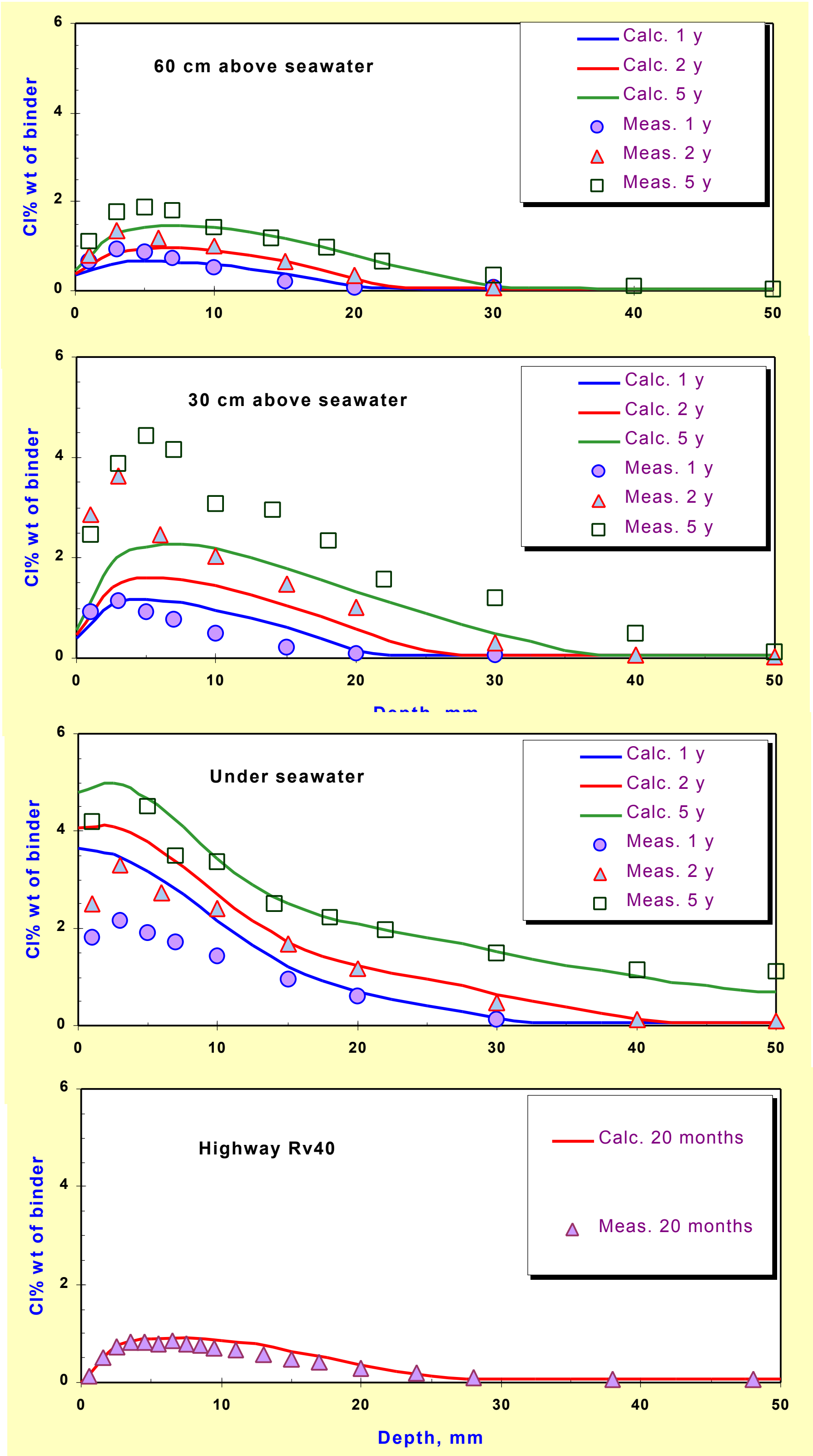


**Figure 8. Example of the predicted chloride profiles. SRPC + 5%CSF, w/b 0.50.**

## 5 CONCLUSIONS

The latest modification has made the model ClinConc applicable to both the marine environment, including submerged, splash and atmospheric zones, and the road environment using de-icing salt in the winter. The verifications up to five-years marine exposure data and two-winters road exposure data show that the predictions of chloride penetration into concrete structures are, in general, fairly well in agreement with the measured chloride profiles.

The exposure environment can be described by the combination of temperature and concentration functions. The former can be expressed as a sine function, while the latter expressed by a statistic normal distribution function. With such a combination, the chloride ingress into concrete under various environments could be approximated by diffusion in a saturated pore system, thus the actual wetting-and-drying processes could be skipped.

There is an urgent need to develop some simple methods for characterising different exposure environments.

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# Combining Artificial Weathering With Chemiluminescence For Lifetime Predictions Of Polymeric Materials

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**Summary: While artificial weathering is a long established tool to investigate long-term behaviour of polymers at outdoor conditions the use of chemiluminescence as a highly sensitive and broadly applicable sensor for the effected change is a recent addition. This combination of artificial weathering with chemiluminescence could already be successfully applied to differentiate and evaluate long-term stability of various real use polymeric building materials relatively to each other and promises to be a useful tool for a variety of other polymeric materials as well.**

**Keywords. Chemiluminescence, lifetime assessment, weathering, polymer.**

## 1 INTRODUCTION

Artificial weathering testing aims at simulating natural weathering at accelerating conditions. To achieve this acceleration either weathering parameters like temperature or UV irradiance are increased above the mean or time compressed cycling is applied in order to repeat stressing periods more frequently than less stressing ones. For a comparison of different materials artificial weathering typically is conducted until a change in a macroscopic parameter like tensile strength, colour, or gloss is observed. The dilemma consists in the goal to achieve information on weathering stability in ever shorter exposure periods on the one hand and still to achieve scalable results to the conditions to be described on the other, with the danger to activate completely different reaction paths and resulting completely different product rankings always at stake.

One way out of this dilemma are more sensitive sensors to detect the changes caused by the weathering, hence allowing a shortening of the preceding weathering duration. A detection mechanism on molecular level will allow accessing changes at the earliest stages of the degradation. Established techniques such as FTIR or NMR also already access this level, but additionally a higher sensitivity of the probing technique is required to detect very few molecules. These requirements are met by chemiluminescence. Compared to for instance FTIR at least two to three orders of magnitude in sensitivity can be gained by using chemiluminescence as the detector for change.

Coming back to the material to be investigated, polymers, in order to explain the molecular detection mechanisms, the question pursued for most outdoor applications is the long-term stability at outdoor conditions.

### 1.1 Degradation mechanism in polymers

Most polymers are liable to oxidation, both photo- and thermo-oxidation principally, but photo-oxidation at least is more damaging for temperatures within the solid state of the polymer. UV radiation can cause the polymeric backbone to be broken up; an initial bond is opened and radicals form on both sides of the fracture. These can either react with themselves or with oxygen. Once initiated the auto-catalytic oxidation process can start to infect the whole material, chains are broken down in ever smaller pieces (or, depending on conditions, also crosslinks can be formed), with ever higher oxidation states; the final state would be carbon dioxide and water. Macroscopically the material looses its toughness and becomes brittle.

As polymers with sufficiently high photo-oxidative stability from an intrinsic protecting chemistry would be extremely expensive, mostly the approach to blend photo-absorbers and stabilizers in sub-percent weight ratios into the polymeric material is been taken. To investigate the long-term outdoor performance of a polymer on molecular level is the question of its stabilisation against this exposure. This stabilisation finally is the result of the interaction of the polymer-stabilizer system

### 1.2 Detection of degradation with chemiluminescence

Chemiluminescence links the rate of the molecular oxidation reaction with a macroscopically detectable emission of light (Ashby (1961), Schard & Russel (1964a), George (1981), Billingham et al. (1988), Matisová-Rychlá. & Rychlý (2000)).

The macroscopically detectable flux of photons, I, is linked to the oxidation reaction with a rate ? that is going on at molecular level by the following equation

$$I = G\,\Phi n$$

which uses G (0 = G = 1) as a constant that incorporates all factors influencing the detection efficiency for photons being emitted, and F (0 = F = 1), which is the overall quantum efficiency of the reaction, defined as the ratio of actually emitted photons per number of potentially reacting particles. This quantum efficiency for chemiluminescence typically is very low, $10^{-3}$ up to $10^{-8}$. It still is possible to detect the weak emission of light through the sensitive detection by single photon counting (G close to a value of 1) and the fact that the photon flux increases linearly with the number of reactive particles. The difficulty to determine the respective quantum yield of a system under evaluation (every slight change in the composition spells a new system) means that chemiluminescence results are not absolute but usually are used relative to rank the performance in residual stabilisation of different materials.

Most undesired irreversible ageing processes in polymers are connected to oxidation. At least for polyolefins this is via the so-called free radical autoxidation mechanism (Scott (1965); Ranby & Rabek (1975); Kamiya and Niki (1978); Stivala et al. (1983); Al-Malalaika (1993)). It allows chemiluminescence to be very widely applied for lifetime assessments (Ashby (1962), Schard & Russel (1964b), Kron 1996, Kohler & Kröhnke (1999), Schartel et al.(1999)). Also, as it is directly linked to the oxidation reaction, it gives a more direct evaluation of the residual stabilization of the polymer than other techniques that often only allow to access a value as result (such as molecular weight, carbonyl index...) the functional relationship of which to the stabilization first having to be assessed.

While the degradative changes effected by the weathering in the polymer mostly are photochemical or photo-oxidative in nature, the investigation of residual stabilisation with chemiluminescence is thermo-oxidative. However, this procedure can be justified, as radicals are the common intermediate for these mechanisms and because radicals are the particular species dealt with by the stabilizer.

The chemiluminescence assessment following the weathering exposure is applied to determine the amount of effective stabilization still active in the polymer. The experiment also could be carried out combining exposure and determination of residual stabilisation in just one chemiluminescence run but this bears various severe disadvantages. Therefore, the first variant was applied.

Examples are presented of relative lifetime assessments for polyethylene films used as greenhouse coverings that were exposed to outdoor weathering conditions under the influence of different stabilizers and of varied weathering conditions.

### 1.3 The material investigated

Agricultural films for greenhouses mostly consist of LDPE films stabilized with HALS (*Hindered Amine Light Stabilizer*). The main purpose is to protect the growing crop in the greenhouse against pests and fungi and to ensure environmental parameters for optimal growing conditions. While at usual outdoor conditions the HALS stabilizer would provide sufficient protection against degradation caused by UV irradiation, in the case of greenhouse films lifetimes are remarkably reduced and far below demands. With hundreds of thousands of tons of greenhouse films being used all over the world, a majority of these only survive one vegetation period before it is likely to fail and is dumped. The degradation mainly consists in an embrittlement of the material at the exposure conditions that causes the material to structurally fail, which then means to the crops in the greenhouse that they are no longer protected. The shortening of lifetimes compared to other uses of the polymer mainly is due to an additional chemical exposure, that has to be taken into account in the form of agrochemicals being used within the greenhouse, as functionalising chemical agents within the film material itself, or as acid rain from external emittents. The widespread heavy use of agrochemicals comprises the burning of sulphur leading up to the formation of sulphur dioxide, which readily oxidises to sulphuric acid, $H_2SO_4$. At the conditions prevailing within the greenhouse the initially diluted acid that forms within condensation dew droplets can concentrate ($H_2SO_4$ only has negligible vapour pressure) to result in highly damaging acid. In this concentrated form, the acid can both attack the polymer itself and the stabilizer system.

### 1.4 The actual measurement program

In this study a specially modified artificial weathering test, the 'Acid Dew and Fog Test' (Schulz et al. (1999)), ADF test, was applied to simulate the climatic conditions of greenhouse films at accelerated conditions and with special consideration of the acid deposition. Subsequently chemiluminescence was used to characterise changes effected by this exposure and to rank different polymer / stabilizer systems according to their residual stability.

## 2 EXPERIMENTAL

All measurements were done in BAM, the Federal Institute for Materials Research and Testing, Berlin, Germany.

### 2.1 Weathering

#### 2.1.1 Equipment

Artificial weathering was carried out in a commercially available weathering device, which is equipped with fluorescent lamps (Weiss Umwelttechnik, Germany; type ‘Global UV Test’). Spectral irradiance was according to ISO 4892-3: 1994 (with no UV output below the normal solar cut-off of 290 nm). The set of UV lamps used to simulate the desired shape of the spectral distribution provides an UV irradiance of 45 W / $m^2$ in the range 290 to 400 nm. The ‘Global UV Test’ device allows running the test at defined, reproducible weathering conditions. Temperature is held constant within ± 1 K and relative humidity within ± 5 %RH.

#### 2.1.2 Test conditions

The 24h ADF weathering cycle comprises:

- a 14 h dry period at varying climatic conditions (9 h at 35 °C and 30 % RH followed by 5 h at 60 °C and 5 %RH),
- a 4 h rain period (demineralized water at 35 °C) at 60 °C and 5 %RH, and
- a second 6 h dry period at 35 °C and 30 % RH.

Five out of seven cycles per week are started with a short-term spraying of a dilute (pH1,5) acid mixture of $H_2SO_4$, $HNO_3$ and HCl in the weight ratio 1 : 0,3 : 0,17 and a pH of 1,5 (resulting in about 3 mg acid solution per $cm^2$ of exposed specimen surface).

To assess the contribution of the acid deposition to the extent of photo-ageing acid free weathering was performed additionally. The cycles of the acid free weathering only differ from the ADF cycles described above in that the acid spraying was dropped.

### 2.2 Chemiluminesence measurements

#### 2.2.1 Equipment

The Chemiluminescence set-up used consists of a own-build heating sample chamber and commercial equipment for the single photon counting detection of emitted photons.

Heating is done electrically, controlled by a Lake Shore (Westerville, OH, USA) 340 temperature controller. This is done within an own-build sample chamber that has a gas inlet and outlet, the sample in its middle, and a quartz window on its top connecting it to a photomultiplier.

An outer, gas tight, stainless steel chamber contains a gas inlet in its bottom. From there the gas from outside first is fed through a chamber filled with steel wool that is situated below a heater and serves to pre-heat inflowing gas. Within the outer chamber, a ceramic inner part is placed, which is thermally insulated from the outer stainless steel part but is connected with it via the gas inlet. Within this inner chamber a pack of resistors is used to heat (room temperature up to 490 K) a removable aluminium pan of 25 mm in diameter from below. Also a PT100 resistor is placed on the bottom of the aluminium pan to measure the temperature that is further processed by the temperature controller and read out from there by a PC. The cover of the inner ceramic chamber consists of a ceramic cover, which can be screwed onto the body of the inner chamber and which incorporates a quartz window in its middle. The quartz window has a hole in its middle to allow the gas to flow outwards. A spring, which is squeezed by the quartz window cover from above, presses the sample close to the aluminium pan. The outer chamber is covered by a removable stainless steel plate that contains another quartz window in its centre (no hole). The gas outlet is below the cover. The cover again is connected as shortly as possible to the housing of a side-on low-noise photomultiplier tube (HAMAMATSU, Herrsching, Germany, Type R1527P select) which is to ensure a maximum in geometrical detection efficiency.

All materials used for the chamber (especially those in 'view' of the photomultiplier) were tested before to be as low emissive in terms of photons released at the temperatures applied as possible.

The signal of the photomultiplier tube is processed by an Perkin Elmer / EG&G/ ORTEC (Oak Ridge, TN, USA) single photon counting device consisting of pre-amplifier VT120C, discriminator 935 and counter 994.

The ORTEC counter and the Lake Shore temperature controller are connected via an IEEE bus to a PC that uses an own programmed Turbo Pascal software to read out and to control.

#### 2.2.2 Measurement Parameters

Before each measurement of a sample, a blank measurement was carried out with all parameters and the entire set-up as in the actual measurement only without sample on the aluminium pan. The obtained background intensities never exceeded limits that would make it necessary to use them to correct the actual sample measurement. However, this procedure mainly was used to ensure that inner part of the cell did not show any contamination from preceding measurements.

The heating program for all measurements presented consisted of 5 min at room temperature to determine the dark rate, then under constant flow of nitrogen heat-up to 172°C (445 K) at a rate of 5 K /min. After the maximum temperature was reached the gas flow was switched to compressed air and temperature was held constant within less than ± 0.1 K error. The investigation temperature is above the melting point of the PE-LD.

Registration of photons is done as integral emission without spectral resolution. The spectral characteristics exclusively are determined by the photomultiplier (within the UV/VIS range of concern to this discussion). The two quartz windows in the light path only show a negligible influence on the spectral transmission behaviour. A postulated emission of singlet oxygen in the NIR region in the course of the oxidation would already be beyond the spectral sensitivity of the PMT used.

Registration of photon rates was carried out continuously (gate time 1 s); count rates were accumulated to a constant integration time of 4 min for each measurement point.

### 2.3 Other Measurements

#### 2.3.1 Tensile Tests

These tests were carried out using a Monsanto T2000 tensometer. Tests were carried out in adaptation of the DIN 53504 standard; tension speed was 100 mm/min. For each measurement five samples were tested. Only the orientation parallel to the direction of extrusion was tested.

#### 2.3.2 Fourier Transform Infrared Spectroscopy (FTIR)

A Nicolet FTIR spectrometer, NICOLET NEXUS FTIR, (Nicolet, Madison WI, USA) was used. Measurements were done in direct transmission mode. Spectral resolution was 4 $cm^{-1}$. For background and sample measurements always 32 scans were recorded. The baseline was smoothed and $CO_2$ and $H_2O$ bands removed by subtraction of the blank spectrum.

A FTIR spectrum of Hostavin N30 shown in 'Fig. 3' was measured in Tetrachloro methane. To match it with the actual peak position of the stabilizer in polyethylene a comparison of unexposed polyethylene with high stabilizer concentration showed that it had to be shifted by 7$cm^{-1}$.

### 2.4 Materials

The material used as an example to compare traditional detectors of the change with chemiluminescence is a polymer, PE-LD, that is stabilized against photo-oxidation with a HALS. Different of these stabilizers blended into the same polymer are to be compared in terms of their protective effect to the polymer. It is to be used as protective cover of greenhouses, typically in Mediterranean countries.

A PE-LD (low density polyethylene) film of 200 µm thickness (± 10%) for use as agricultural films was investigated, containing 0,2% and 0,4% of stabilizers, respectively. Two different HALS materials were compared within the PE: Hostavin N30 and Chimasorb 119. The film was especially prepared not to contain other additives.

The material was characterised using differential scanning calorimetry (DSC), UV/VIS and FTIR spectroscopy.

Material exposed to the ADF weathering without acid will be labelled 'w', that one exposed to the ADF test including acid will be labelled 'a', unexposed material 'u'.

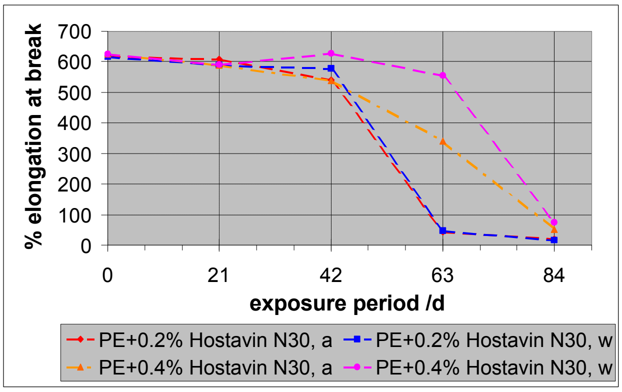
## 3 RESULTS AND DISCUSSION

In order to show why a sophisticated technique as chemiluminescence could be useful firstly it shall be explained what shortcomings the traditional techniques, like tensile test of FTIR spectroscopy, suffer from. Therefore, in the following a preceding weathering on the same samples was to be compared with different detection techniques.

### 3.1 Detection through Tensile Tests

The property of practical importance is the toughness of the polyethylene greenhouse films. The technical property to test for changes effected by the weathering exposure therefore used to be the elongation at break tested in a tensometer after the weathering.

Examples of tensile tests of the material are depicted in ‘Fig. 1’ for different concentrations of the stabilizer Hostavin N30 in PE-LD and in ‘Fig. 2’ as comparison of the performance of Hostavin N30 with Chimasorb 119.



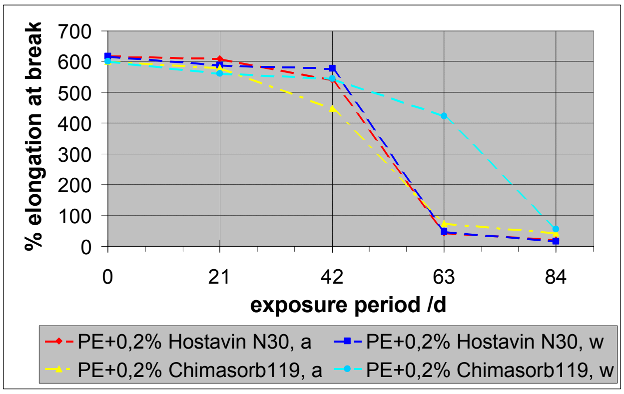




**Figure 1. Tensile test of LDPE with 0,2 and 0,4 % Hostavin N30 stabilizer after 0, 21, 42, 63, and 84 days of artificial weathering without ('w') and with ('a') acid precipitation**

**Figure 2. Tensile test of LDPE with 0,2 % Hostavin N30 and Chimasorb 119 stabilizer after 0, 21, 42, 63, and 84 days of artificial weathering with-out ('w') and with ('a') acid pre-cipitation**

The results can be summarised as follows:

- The results of tensile measurements show a comparatively high measurement uncertainty.
- In terms of exposure with and without acid precipitation ('Fig.1') 0,2 % Hostavin N30 does not show a difference, while the difference for 0,4 % becomes quite remarkable. Acid precipitation causes a deterioration of the tensile behaviour.
- A differentiation between the two stabilizers ('Fig. 2) grows after 21 days of weathering, reaches a maximum after 63 days and is gone again at 84 days.
- The performance of Chimasorb 119 already is at 0,2 % about the same as that one of Hostavin N30 at 0,4 % ('Fig. 2').

This allows the following conclusions to be drawn:

- In terms of testing: Samples have to be taken and tested after 21, 42, 63 and 84 days if tensile tests are to be used as sensor for the effect of weathering on polyethylene. In this case, only the measurement window between 42 and 63 days could be used to make a differentiation. The bad signal/noise ratio for the measurement of low values of elongation at break as occurring for high duration of weathering give tensile tests a high degree of uncertainty in the evaluation of different samples.

In terms of degradation mechanism:

- Degradation starts to show macroscopic effects after an induction period of about 21 days, increases and reaches saturation and an end value after 84 days of weathering.
- The fact that for Hostavin N30 a differentiation in response to acid precipitation compared to weathering without acid only shows for the higher stabilizer concentration could mean that 0,2 % of the stabilizer are not enough to leave effective stabilizer protecting the polymer after 21 days. If there is enough stabilizer remaining in the polymer (as for the case 0,4 %) it is liable to acid attack. Also, the occurrence of differentiation for 0,4% might be a hint to an increased stabilizer liability to acid attack.

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## 3.2 FTIR detection

‘Figure 3’ summarises the results of weathering with ('a') and without ('w') acid precipitation for 21 and 42 days for the system PE-LD and Hostavin N30 as detected by FTIR.

The results of the FTIR analysis can be characterised as follows:

- Oxidation can be detected as an increase of the broad carbonyl band in the region 1680 - 1820 $cm^{-1}$.
- A competing, neighbouring band 1680 - 1720 $cm^{-1}$ owing to the stabilizer 100% 'Hostavin N30' curve compared to 0% for PE unexposed without stabilizer (‘unstab., 0d_u’) is so much dominated by the PE oxidation band that it can not be used to determine the concentration of the stabilizer.
- The increase in the carbonyl band is not linear with duration of weathering. It rather exhibits an induction time of between 16 days (still stabilizer, hardly carbonyl) and 21 days (first carbonyl). From then on it rises and shows a considerable differentiation between much more deteriorating acid exposure compared to weathering without acid precipitation.
- A broad range of different oxidised species produces a varying distribution of the resulting peak in the carbonyl region ('Fig.3', 42d_a curve compared to 42d_w curve), which makes comparisons difficult.

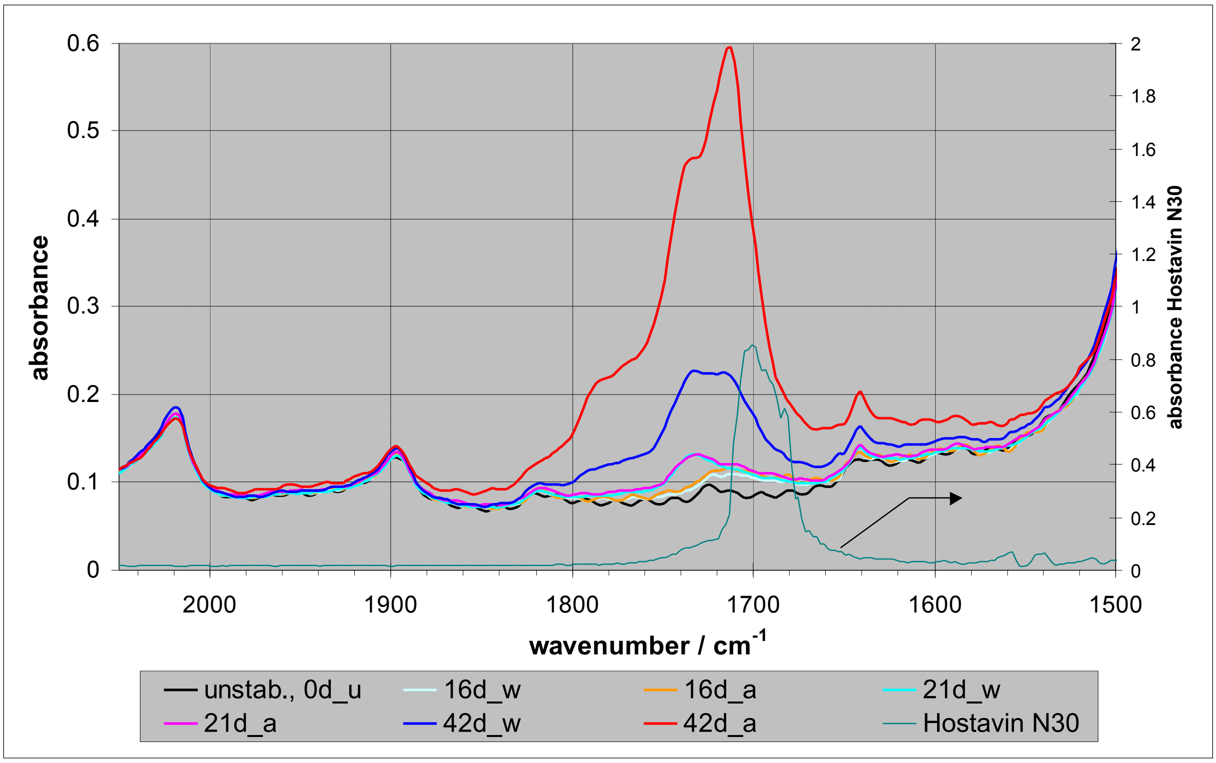


**Figure 3. FTIR analysis of the effects of artificial ADF weathering for 16, 21 and 42 days on a LDPE film stabilized with 0,2 % Hostavin N30 and pure Hostavin N30 (‘Hostavin N30’) as well as unstabilized unexposed material (‘unstab., 0d_u’) as comparison. Labelling: days of weathering, followed by mode of weathering (‘u’ = unexposed, ‘w’ = ADF without acid, a = ADF with acid )**

This brings up the following conclusions:

While FTIR has got undisputed virtues in identifying chemical processes and species the main difficulties for a potential use to result in a ranking of polymer stabilizer systems arises from the following facts:

- FTIR sensitivity is too low for the initial stages of the oxidation (in this case: anything during the first 21 days of weathering).
- A monitoring of the extent of oxidation by the growth of the carbonyl region with the aim to compare it to another stabilizer system becomes very subjective as to the selected area that is being evaluated and by overlapping different groups.

### 3.3 Chemiluminescence detection

Ahead of the actual measurements a comparison of the effects of a change in the temperature used to carry out the chemiluminescence experiment is to demonstrate a few of the basic features of chemiluminescence itself and of the dependence of the characteristics of the resulting curves on this parameter in particular.

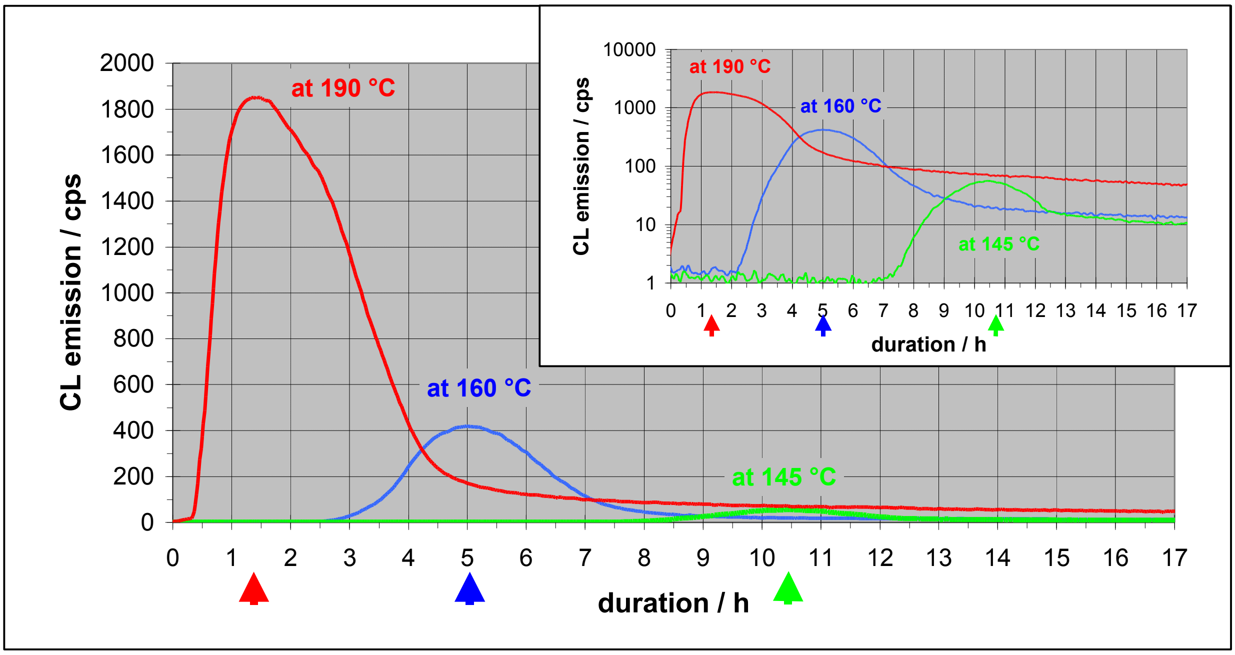


**Figure 4. CL investigation of the influence of temperature. PE-LD stabilized with Hostavin N30; CL under air, isotherm conditions. Insert: semilogarithmic presentation. Times needed to reach maximum are indicated by triangles.**

The following basic features of chemiluminescence (CL) can be deducted from 'Fig. 4' and can be generalised for other CL measurements:

- The CL emission over time shows a sigmoid shape.
- The oxidation starts with an induction time (curves for 160 and 145°C) during which initiation and consumption (stabilizer) of reactive radical species are at stationary low level.
- After the end of the induction time the oxidation accelerates and spreads over the whole material.
- The maximum reached corresponds to a maximum in the rate of the oxidation reaction.
- After the maximum the CL emission asymptotically approaches a stationary end value.
- With an increase in the temperature used for the CL experiment the emission maximum shifts to a longer duration (increase in induction time) and the height of the maximum increases exponentially. (Also the stationary end value or emission increases with temperature).
- A change in the shape as occurring for the 190 °C CL curve hints to potential different reaction paths being thermally activated at higher temperatures

Additionally it can be generalised that a high stability shows as long induction time and time to reach the maximum and as low chemiluminescence intensity.

The tuning of the optimal temperature to be applied for the CL experiment therefore must find a compromise between higher temperature to achieve an acceptable duration of the CL experiment on the one hand and lowest possible temperature not to reach thermally activated states that are not relevant for the conditions to be characterised on the other.

The actual CL measurement for PE-LD stabilized with Hostavin N30 in respect to the effects of preceding weathering including acid precipitation is presented in 'Fig. 5'. For comparison, the respective CL measurement for an unexposed sample is shown.

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- The time to reach the maximum decreases and the maximum increases with the duration of weathering. After 21 days of weathering the height of the maximum decreases again (competing yellowing of the sample).
- In the time to reach the maximum there is a noticeable reduction between 16 and 21 days of weathering which can be explained in a change in the degradation mechanism.
- While a maximum in intensity is reached for a exposure period of 21 days it decreases again for an exposure of 42 days.
- The exposure period causes a systematic increase in the slope of the respective CL curves for the first period.
- A comparison for the performance of the two stabilizers Hostavin N30 and Chimasorb 119 in PE-LD is given in 'Fig. 6' for 16 days of weathering:
- Already for this exposure period, CL allows to make a differentiation in the performance of the two stabilizers.
- PE-LD stabilized with Chimasorb 119 in all cases (unexposed, weathered without acid precipitation for 16 days, weathered with acid precipitation for 16 d) shows higher stability than PE-LD stabilized with Hostavin N30.
- While the stability of unexposed PE-LD stabilized with Chimasorb 119 is outstanding the same system shows strong effect to all kinds of weathering, with the weathering including acid precipitation showing the strongest destabilizing effect.
- For PE-LD stabilized with Hostavin N30 the effect of weathering with added acid precipitation only just shows in comparison to weathering without precipitation.

## 4 CONCLUSIONS

- Chemiluminescence allows to directly evaluate the relative residual stabilization of a polymeric system that determines the lifetime of a polymer.
- It is more sensible than the other techniques compared.
- A particular sensitivity for the first stages of an oxidation reaction allows an early differentiation of different systems in terms of their stabilisation against oxidation.
- The higher sensibility, apart from the detection principle, could be due to another procedure (e.g. another effective dimension) the technique counts degraded species.

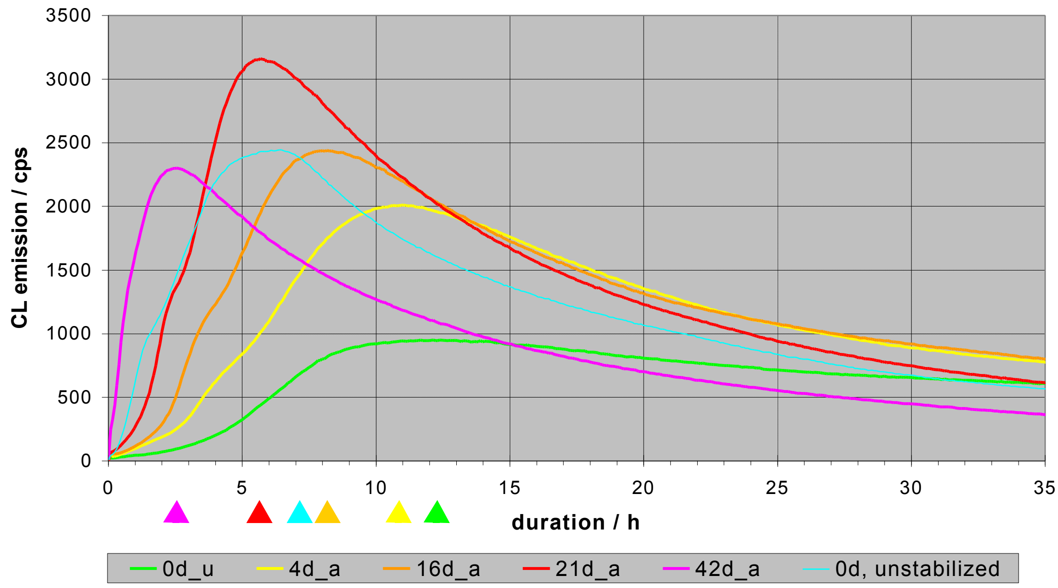


**Figure 5. CL investigations at 172 °C for unstabilized unexposed PE-LD ('0d, unstabilized') compared to PE-LD stabilized with 0,2 % Hostavin N30 that remained unexposed ('0d_u') or was weathered including acid precipitation for 4d, 16d, 21d and 42d. Times to reach maximum indicated by triangles.**

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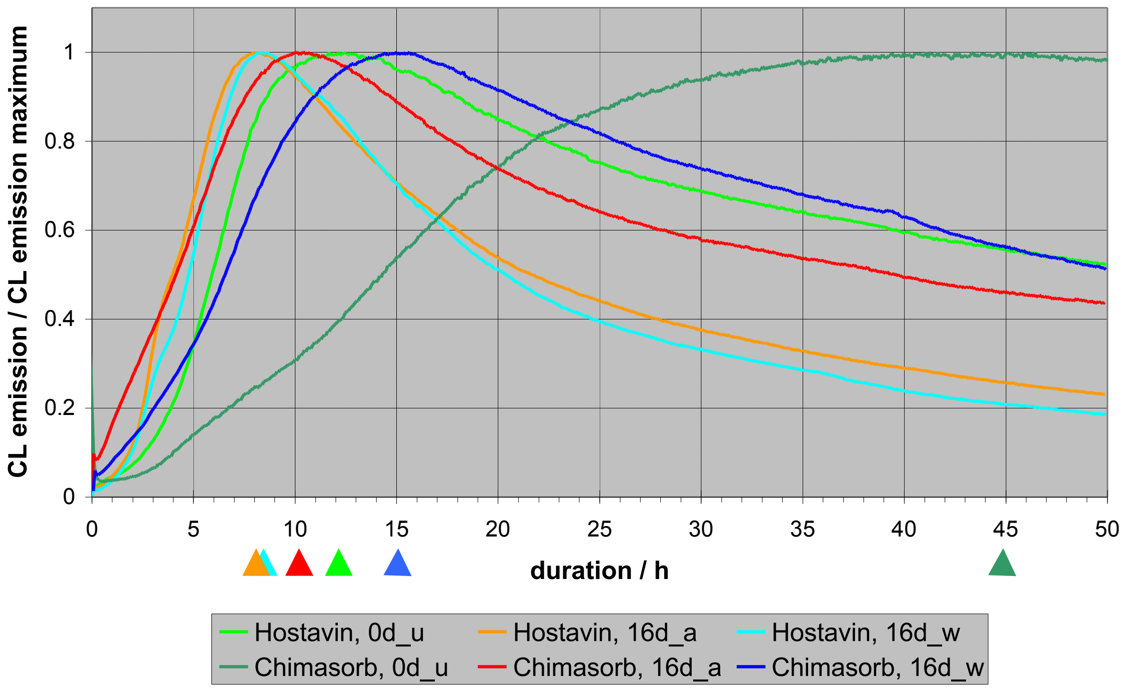


**Figure 6. CL evaluation at 172 °C of the performance of PE-LD with Hostavin N30 compared to PE-LD with Chimasorb 119 as maximum normalised emission as a function of CL duration. Days given (0d=unexposed, 16d) relate to respective exposure period. Weathering either with acid precipitation ('a') or without ('w').**

## 5 OUTLOOK

Initial experiments with polycarbonate roof materials that were weathered in BAM and characterised in CSIRO indicate a systematic CL response to preceding exposure and promise to complement FTIR and other results (Schulz, U. & Tjandraatmadja (2002)) of the effects of weathering on this material. In addition, these CL experiments may offer further insight into the degradation mechanism.

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# Predicting The Service Life Of Natural Stone

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**Summary: This paper covers research carried out as part of a major project funded by the UK Department for the Environment, Transport and the Regions (Contract cc1600 – 'Service Life Prediction for Natural Stone) between 1998 and 2001. The aims of the project were to provide assessments of the decay of materials and the economic implications of both the damage and the remedial actions. The results were used to give a broad knowledge of service life prediction and so to underpin the development of a methodology for whole life costs. This approach will allow a wide range of existing and new data to be used at BRE and by others in predicting the service life of new stone and stone constructions.**

**The project has included original research on areas such as the effect of de-icing salts and vehicle emissions on natural stone as well as collaboration with a wide range of international projects.**

**The key concept is that 'risk' is the common currency (rather than monetary value). Thus, the primary output is the estimation of risk. The choice of materials, their exposure and use lead to some level of risk (which includes financial risk). The acceptability of this risk is determined within the context of desired criteria and restraints. Thus, the outcome is a weighted risk analysis which combined with a risk threshold gives a success/ failure envelope.**

**Each failure mode has a specified probability associated with it between 0 – 1 which relates to the material properties and the environmental factors associated with the location of the material. These are based on either known data sets for all UK stones (or a reasonable guess), or a specific measurement made on that stone type and available environmental data.**

**Coupled to this risk is a user defined 'importance'. Here the user is invited to define the levels of importance attached to particular characteristics.**

**The main framework of the integrated expert system has been set up and is coded into Visual Basic. This represents the 'backbone' of any specific embellishments in that it proscribes a method of synthesising materials, usage and risk into a coherent assessment. Work is continuing under a new contract that will allow refinement of the existing model.**

**Keywords. Service life, natural stone, deterioration, risk assessment**

## 1 INTRODUCTION

This paper covers research carried out as part of a major project funded by the UK Department for the Environment, Transport and the Regions (Contract cc1600 – 'Service Life Prediction for Natural Stone) between 1998 and 2001. The function of the project was to predict the service life, both from new and residual, for natural stone units and components used in the construction industry. The project fell within the priorities set-out in the 1997-98 UK Government Technology and Performance Business Plan. The results from the project are being used to enhance the service life and use of natural stone in construction. The project began in April 1998 and was completed in March 2001.

The aim of the project was to provide a series of assessments of the decay of materials and the economic implications of both the damage and the remedial actions. The results are being used to give a broad knowledge of service life prediction and so to underpin the development of methodologies for whole life costs.

The project required a wide range of existing and new data to be used by BRE in predicting the service life of new stone and stone constructions.

The final objective in the project was to integrate the data from a range of associated research tasks into an experet system model and provide data on the service life and residual life of natural stone used in construction, including data on costs of maintenance of stone during its service life, guidance on maintenance and on the advisability of using surface treatments. The research tasks covered the effect of acidic gases and particulates, long term climate change, freeze/thaw cycles, deleterious materials in the stone (for example clays), de-icing salts, and cleaning and repair.

The purpose of this paper is to summarize the work that been carried out to date in the development of the expert system which integrates the research findings.

## 2 OUTLINE OF THE EXPERT SYSTEM MODEL

### 2.1 Key concepts

The key concept is that 'risk' is the common currency (rather than monetary value). Thus, the primary output is the estimation of risk. The choice of materials, their exposure and use lead to some level of risk (which includes financial risk). The acceptability of this risk is determined within the context of desired criteria and restraints. Thus, the outcome is a weighted risk analysis which combined with a risk threshold gives a success/ failure envelope. These concepts are summarised in Fig 1.

The links between the various blocks have been defined and their implementation is described below.

### 2.2 Main framework of the analysis

A design criteria for gathering and displaying information arising from the investigation of a particular material is that the minimum information should be requested or displayed in order to make the necessary estimation of risk. Within that constraint there should be sufficient information available should the necessity to 'dig deeper' arise. To enable this the general structure of the system will be hierarchical - progressing from the simple towards more complex or detailed information.

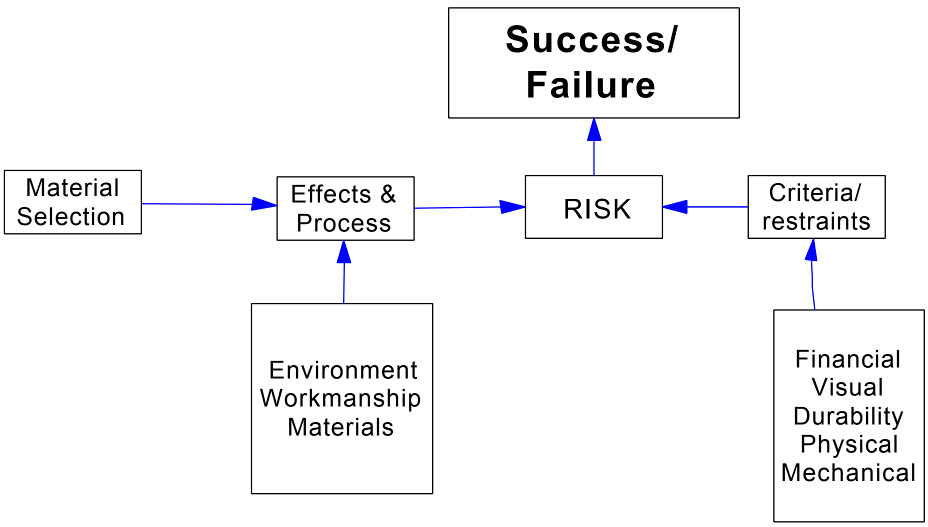


**Figure 1 Flow of information towards a risk criteria**

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The main blocks of the analysis are show in Fig 2 (below).

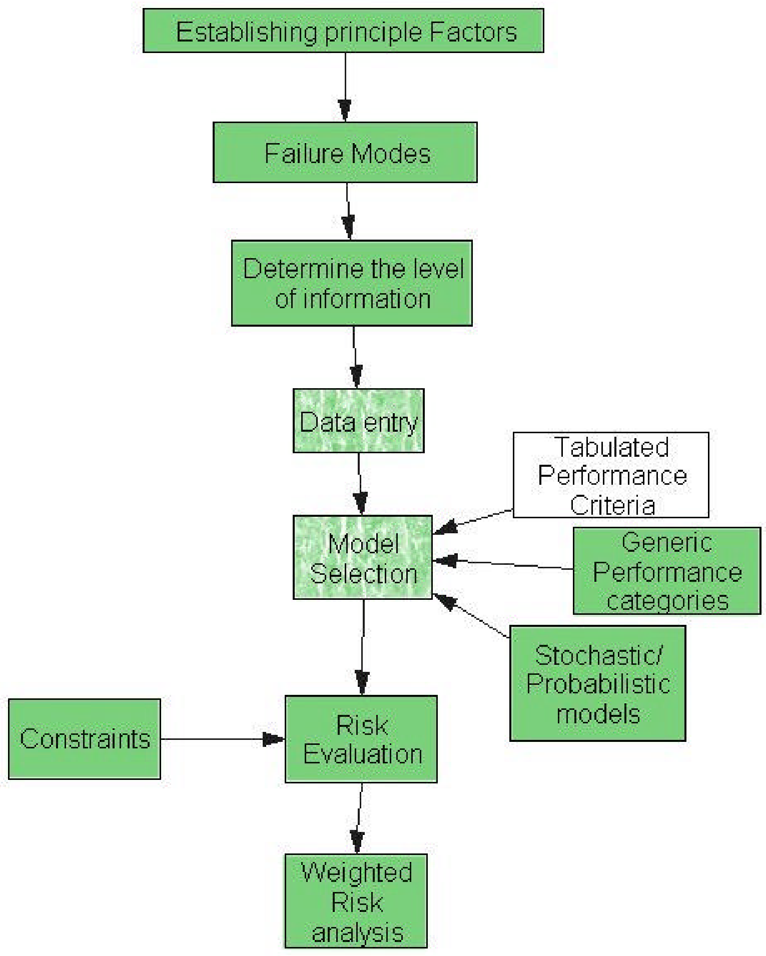


**Figure 2. Flow diagram for the analysis**

Boxes coloured ‘green’ have been specified and implemented. Green textured boxes have been specified and those in white require more work.

The first three boxes have been implemented as a simple selection procedure where the ‘user’ is invited to select entries from a series of scrolling lists. These lists are updated with every selection so that only the minimal information is requested from the user. This is based on the matrix system described elsewhere. An example of the selection procedure is given below in Fig 3.

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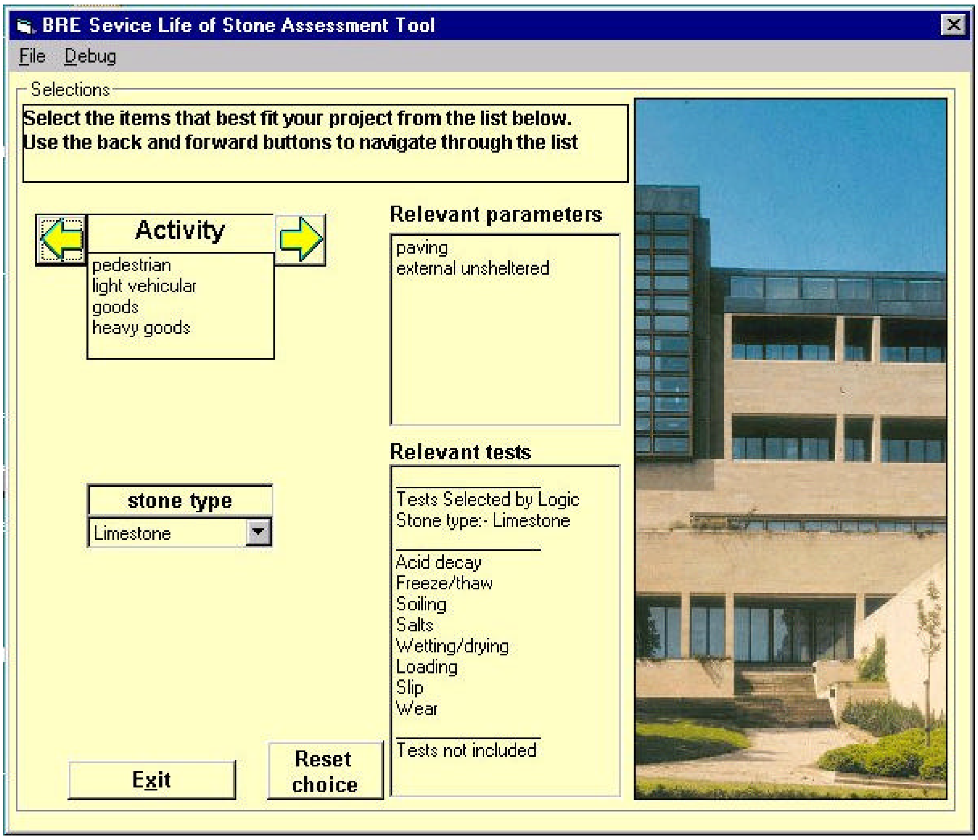


**Figure 3. Example of the selction procedure screen**

The output from this module are the tests relevant to the conditions and environment selected by the user. At this point the user is invited to enter whatever data they have available for that material. It is envisaged that the entry screen will be similar to the example show in Fig 4.The probability of failure is calculated from these inputs. These probabilities are coupled to a user defined 'importance'. This defines a series of variables with a range from 0 to 10 which are linked to various failure modes via a matrix-

| | Decay | Fracture | Load Failure | Discoloration | Movement | Wear | Slipperiness |
|---|---|---|---|---|---|---|---|
| **2.3 Acid decay** | X | | | | | | |
| **Freeze/ thaw** | X | | | | | | |
| **Soiling** | | | | X | | | |
| **Salts** | X | | | X | | | |
| **Wetting/drying** | X | | | | X | | |
| **Loading** | | X | X | | X | | |
| **Slip** | | | | | | | X |
| **Wear** | | | | | | X | |

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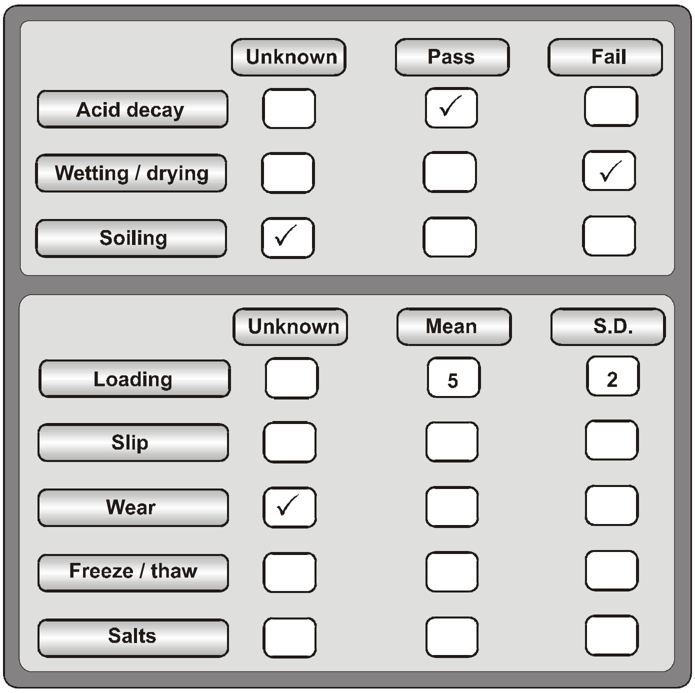
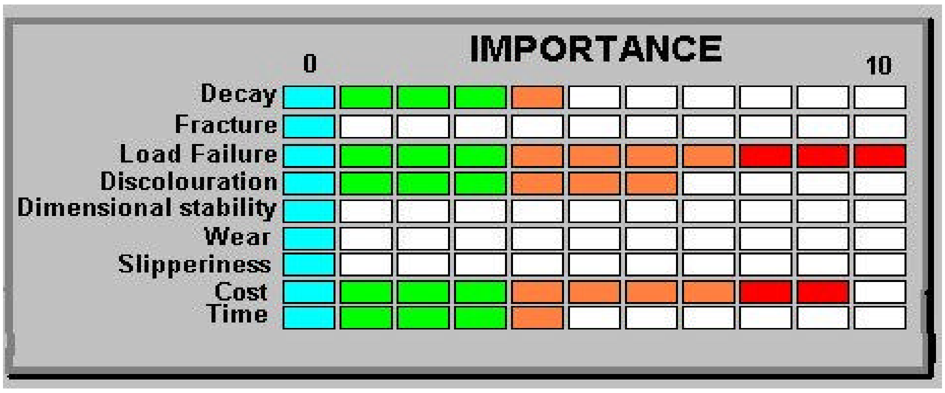



**Figure 4. Example of the proposed entry screen.**

It should be noted that the risk factor associated with loading failure is always 10 (i.e. load failure is not an acceptable risk in any form). From these two pieces of information, the probability and the relative importance, a risk factor can be assigned - the product of ten times the probability and the 'importance' rating. This creates a scale of 0 to 100 which measures the relative risk. Columns with multiple entries like Decay will display the maximum risk selected from items in the column.

The combination of the probability and users criteria are combined to present an overall risk (see diagrams below).




From here the user is invited to re-consider any of the inputs should the risk be deemed to be too great for any particular item. Suitable help screens will support this reiteration process.

## 3 DEVELOPMENT OF CALCULATIONS

### 3.1 Determine the level of information

The inference of risk can proceed from several different levels of information - factual/measured, generic estimation, or opinion. The latter two categories lie in the area of subjective analysis with the generic information perhaps bridging between the factual and opinion. These two can be specified by categories such as 'High', 'Medium' or 'low' which are amenable to analysis using 'fuzzy' logic to give categorised outputs. Moving towards the factual/measured information, the generic information could be refined by an approximate knowledge of the probability distribution via the shape and likely range of statistical parameters. These lend themselves to defining the most likely outcome as a combination of the statistical parameters. The final level of detail is that of actual measurement and the associated uncertainties (variances/ Standard deviations) which in a sense is a refinement of the generic statistical analysis.

### 3.2 Data input

This process follows on naturally from the definition of the available data. Again the structure will be such that the inputs screens are filtered to provide the necessary level of input.

### 3.3 Model selection

The models will be defined by three separate branches :-

- Tabulated/performance
- Generic Performance
- Statistical/Stochastic

This process is analogous to regression analysis where the users is asked what type of curve is to be fitted to the data. The implicit suggestion is that as the user requests higher and higher levels of regression (i.e linear to polynomial) that there is more confidence in the accuracy and detail of the data. Here also the successive levels of refinement are determined by the amount of information that the user has at their disposal and the depth of analysis required. Tabulated/performance being the least onerous and the statistical methods the most accurate.

For the tabulated/performance analysis the analysis is made entirely by comparison with existing data and experience i.e. I have a material which is similar to X which behaves in such and such a manner with risks X, Y and Z.

Generic Performance at the lowest level uses categories specified either by terms such as 'High', 'Low' and 'Medium'. These are combined to give a similarly generic statement on the basis of 'fuzzy' logic rules for output. The next level would be to use approximate statistical parameters to estimate the most likely outcome.

The Statistical/Stochastic model would be a refinement of the approximate parameter model based on actual measured data.

The above methods are considered from the premise that the physical interactions relating the performance are impossible to determined from the precise interaction of all the factors. This is because the factors themselves define a very complex interaction system. Thus, although much is know about many of the interactions, it is not possible to define, even in a crude deterministic fashion, the all the interactions within a realistic cost and time frame.

An alternative to the above approach, building up from a solid deterministic foundation for each eventuality, is to proceed from a position of ignorance gradually introducing more detail as appropriate. One approach for achieving this is to use multiplying factors (a,b,c,d,e,f) to account for the variation that operations or the environment endow on a material (ISO 2000). The factors are defined as : -

| | | | |
|---|---|---|---|
| Factors related to inherent quality characteristics | $A_1$ | Performance of materials | Material type or grade. Manufacture, storage, transport |
| | $A_2$ | | Durability features : Protection system |
| | B | Design Level | Details of construction: joints fixing, incorporation, sheltering by rest of structure |
| | C | Work execution level | Site work : Quality to standard or manufactures specification. Level of workmanship, climatic conditions during the execution work. |
| Environment | $D_1$ | Indoor environment conditions | Condensation, aggressiveness of environment, ventilation |
| | $D_2$ | Outdoor environment conditions | Macro, meso, micro environment conditions |
| Operation conditions | E | In use conditions | Mechanical impact, category of users, wear and tear, vandalism |
| | F | Maintenance | Quality and frequency of maintenance, accessibility for maintenance |

The limitation with this technique is that the product of the factors rapidly diminishes and is sensitive to having one low estimation for one of the factors. Introduction of the weighted mean of the risk terms absorbs the impact of a low multiplicative coefficients.

### 3.4 Risk evaluation

With each output of the model there is an associated a risk or liability and cost. For example, a slippery surface on paving carries a high risk and cost implications yet the same material used in cladding represents a minimal risk due to its slipperiness. The significance of these risks/costs is determined by the imposed constraints. Maintenance of paved surfaces is relatively cheap whilst a similar operation for cladding would be costly. Thus the constraint allows an informed analysis of the risk. It may be that there is a constraint to keep a façade clean from asthetic grounds rather than risk mitigation which would represent a cost liability. However, the combination of the risk and cost give a better indication of the appropriate action. The outcome from this section can then be used to assign a success/failure map on the basis of the constraints with associated cost implications.

Each failure mode has a specified probability associated with it between 0 – 1 which relates to the material properties and the environmental factors associated with the location of the material. These are based on either known data sets for all UK stones (or a reasonable guess), or a specific measurement made on that stone type and available environmental data.

Data from earlier research programmes relating to sandstone and limestone has been used to calculate the probabilities of failure for Acid decay, Freeze/ thaw, Salts, Slip and Wear. For the loading the natural flexural strength and strength after Freeze/thaw testing have been used in accordance with ‘Flooring, Paving and setts’ (Yates *et al*. 2000) and ‘External Cladding’ (Yates *et al*. 1998) to determine the probability of failure. Future developments of the software will incorporate the terminology from both these documents to ensure greater user familiarity. Data for the slates will come from archive material used in the publication “The building slates of the British Isles” (Hart 1991).

These publications represent the dominant use of use of British stone within the United Kingdom and are the only current books dealing exclusively with dimension stone. There are no similar collations of imported material (marble, granite) and little data available regarding British granite. It is intended to supplement this paucity of data with information about the properties of stone from extensive lists appearing in a variety of text books. These tend to concentrate on the engineering properties of the materials and focus on mining, tunnelling and the extractive industries rather than dimension stone.

For the more esoteric variables concerning the appearance of the materials, soiling and staining, there is little hard evidence of the rates of discoloration on the differential soiling due to material type or location. Any such indicators tend to be on generic materials i.e. stone rather than specific building stones. There is some indication that they do soil at different rates but insufficient to determine any meaningful relationship. Similarly for Wetting/drying there is no systematic source of information. Blackening from pollution is described by a few empirical equations which apply equally well (or not at all) to all the materials and have not been tested over a wide geographical region. They at least give orders of magnitude estimates. With respect to biological growth on materials this has been studied even less – there are indications of the time scale (circa 2 – 10 years) but little more with regard to actual rates and prevalence on different stone types. An inverse relationship with sulphur dioxide concentration is conceivably a reasonable starting point but there is little indication of the rate at which colonies are established and become viable in connection with geographical location. Staining by spillage is the least understood with virtually no information.

Thus, whilst quite a lot of information is known there are a number of probabilities for which there are no figures at present. Hopefully, this will be rectified in the course of time.

## 4 FRAMEWORK FOR INFORMATION FLOW

The interaction between the main parameters of the model - Material, Use, Failure mode, Information, and Output - is very complex and difficult to assembly as a customary programmers flow chart. The number of interactions between the different components is very high and this makes it impossible to express coherently on two dimensional paper by either graphical or tabulated methods. Therefore, the first task was to define the vectors.

For the analysis of service life the process is defined almost entirely by the material, use, & environment. These determine the relevant failure mode and tests, leading on to pertinent information and output. Workmanship & process are post-selection which have a bearing on the final outcome rather than the initial selection process.

Defining the vectors and their numeric key (for ease of identification) :-

| **Material** | | **Use** | | **Environment** | | |
|---|---|---|---|---|---|---|
| Limestone | 1 | Flooring | 6 | Internal | Wet | 11 |
| Marble | 2 | Paving | 7 | | Occasionally wet | 12 |
| Sandstone | 3 | Cladding | 8 | | Dry | 13 |
| Slate | 4 | Masonry | 9 | External | Unsheltered | 14 |
| Granite | 5 | Roofing | 10 | | Occasional Unsheltered | 15 |
| | | | | | Sheltered | 16 |
| | | | | Temperature | | 17 |
| | | | | Humidity | | 18 |
| | | | | Elevation | Low | 19 |
| | | | | | High | 20 |

| Failure Mode | Test | |
|---|---|---|
| Acid decay | Acid Immersion | 21 |
| Freeze/ thaw | Freeze/thaw | 22 |
| Soiling | Exposure trial/ growth cultures | 23 |
| Salts | Salt crystallisation | 24 |
| Wetting/drying | Wetting & drying cycles | 25 |
| Loading | Mechanical & physical tests | 26 |
| Slip | Slip (Wet & Dry) | 27 |
| Wear | Abrasion | 28 |

These lead to sub-groups which are dependant on earlier selections

| Loading | | | | | |
|---|---|---|---|---|---|
| Activity | | Usage | | Natural | |
| Pedestrian | 29 | Library/supermarket | 33 | Snow | 36 |
| Light Vehicular | 30 | Department store | 34 | Wind | 37 |
| Goods | 31 | Access | 35 | Water loading | 38 |
| Heavy goods | 32 | | | Thermal | 39 |
| | | | | Moisture | 40 |
| | | | | Structural | 41 |

| Soiling | | Salts | | Wet | |
|---|---|---|---|---|---|
| Biological | 42 | Ground | 45 | Acidic | 48 |
| Mechanical | 43 | De-icing | 46 | Neutral | 49 |
| Staining | 44 | Cleaning agents | 47 | Caustic | 50 |

The numeric key can be used to assign relevant groups of in an array (50 x 50) or 50 element sets. These sets control the route through the structure of the programme.

Looking at the dependencies between the major groups we have :

Failure Mode = f(Material, Use, Environment)

Whilst the remaining terms are dependant on the Use and Environment but independent of the material. This suggests that Use & Environment should be examined as a separate set, the outcome of which combined with the Material indicates the failure mode. This reduces the size of the array to 37 x 37 with an additional array of 8 x 37 for the failure modes.

Enumeration of the sets is carried out on the basis of determining the exclusions and leaving in those elements which are not influenced by the appropriate key word. E.g. Keyword = Dry, Dry excludes Wet but has no influence on the loading, thus the set includes numbers 29 - 35 but excludes 11 & 12. This means that progressive selections eliminate the exclusions but leave a number of keywords that have not been selected. A provision for eliminating these from the selection by direct deletion is required to tidy up the selection set. Once this has been done the failure mode tests can be determined from the resulting selection set.

## 5 DETERMINATION OF THE PROBABILITIES

Where the material properties are unknown the probability is based on the concept that the material could be ‘sourced’ from any of the UK quarries and thus represents a ‘blind’ selection of the material. At present the temperature data within the selection procedure is equally ‘blind’, considering that the material might end up anywhere in the UK. Once the properties of the material and location are known more precisely then a more considered calculation can be made. For illustration data for sandstone is summarised below representing the case for this blind selection.

### 5.1 Acid immersion

This is the proportion of samples that pass or fail the acid immersion test. For the sandstone it is recognised that the probability changes slightly between ‘Red’ sandstone and the ‘Brown’ sandstone. All three figures are relatively close (0.17, 0.2, 0.13 respectively). Given that the failure of the ‘red’ sandstone is somewhat erratic using the general figure of 0.17 is sufficient.

### 5.2 Slip data

For the sandstone slip data the probability of failure is :

Dangerous = 0

Marginal = 0

Satisfactory = 0.513

Excellent = 0.487

The categories are as Yates *et al.* 2000 and all the sandstone are above 40 = satisfactory.

### 5.3 Abrasion

For the sandstone abrasion data the probability of failure is :

Intensive = 1- 0.636 = 0.47

Moderate = 1 - 0.182 = 0.82

Individual = 1 - 0.182 = 0.82

### 5.4 Loading & Freeze/thaw testing

Loading for both cladding and paving is determined by the flexural strength of the material. Generally the compressive strength is more than adequate as it is frequently much higher than concrete. Nevertheless, the compressive strength needs to be considered in some members at the design stage. This is very specific to the actual situation and cannot be considered in the expert system at present (the system is not intended to be a design tool for masonry construction).

Flexural strength is affected by the action for freezing and thawing. There is no systematic change in the strength with increasing material strength yet there is a significant shift in the distribution of the strength in the mid-range region.

This changes the cumulative probability on which the assessment is made. Thus, there are two probabilities - one for fresh material and another for material subject to freeze/thaw.

The probability of failure is determined using the formulae presented in Yates *et al*. 2000 and Yates *et al*. 1998 which requires the thickness, width and length of the material to be specified, plus the observed standard deviation of the flexural strengths. This enables a lower 95% bound on the flexural strength to be set which can be utilised to determine the probability of the actual value falling below this bound.

The probability of failure for fresh material and material subject to freeze/thaw are combined using the probability (Tp) that there is a significant number of days where the temperature is below zero. This data is taken from Lacey (1977).

The information is combined as a weighted average A*Tp+B*(Tp-1), where A and B are the probabilities for freeze/thaw and 'fresh' material respectively and Tp the probability of freezing.

### 5.5 Crystallisation test

There is a relationship between the flexural strength, crystallisation loss and porosity. Below threshold values of these variables there is a greater chance of damage occurring.

$P_{Fresh} = 0.57$

$P_{F/T} = 0.56$

Again the risk from the fresh and the freeze/thaw flexural strength is weighted by the temperature distribution, though in this instance the difference in probability is marginal.

## 6 OBSERVATIONS AND CONCLUSIONS

The main framework of the integrated expert system has been set up and is partially coded into Visual Basic. This represents the 'backbone' of any specific embellishments in that it proscribes a method of synthesising materials, usage and risk into a coherent assessment.

Discussions within BRE have highlighted a number of potential uses for such a system; beyond material specification and selection.

There is a considerable amount of effort required to bring the present ideas to fruition with respect to:

- Collation of data and statistics for other materials,
- Preparation of a clear logical user interface,
- Instructions and manuals for different potential users.

Given these two points it may be that a number of versions of the 'expert' system might exist where by the entry point to the program and presented information are tailored to specific users e.g. material specifiers, suppliers and financial backers who require information on the likely whole life costs and performance.

## 7 ACKNOWLEDGMENTS

The project was carried out on behalf of the UK Department for the Environment, Transport and the Regions who funded the project.

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# Durability Predictions Using Early-Age Durability Index Testing

JR Mackechnie & MG Alexander

**Summary**: **Durability of reinforced concrete structures is often dependent on the corrosion of reinforcement due to ingress of aggressive ions and fluids. Using the premise that the potential durability of concrete is determined by the protection provided by the cover concrete to the embedded steel, the resistance may be defined in terms of transport properties such as absorption, permeation and diffusion. A suite of durability index tests was developed to characterize the early-age resistance of concrete to transport of fluids and ions that affect corrosion of reinforcement. These tests were found to produce reasonable predictions of durability performance for reinforced concrete structures. Early-age laboratory characterization testing is discussed together with field data regarding carbonation rates and chloride ingress into concrete. Findings suggest that the approach may be usefully applied as performance specifications where durability of reinforced concrete structures must be guaranteed.**

**Keywords. Carbonation, characterization, chlorides, durability, predictions**

## 1 INTRODUCTION

The bulk of durability problems concern the corrosion of reinforcing steel rather than deterioration of the concrete fabric itself. The adequacy of the concrete cover layer is therefore critically important in resisting aggressive agents from the surrounding environment. A plethora of durability tests has been developed to measure fluid transport rates by various mechanisms through concrete. Most methods require sophisticated equipment and complex monitoring and therefore have limited practical value for site concrete.

The concept of durability index testing was proposed to provide practical means for characterizing the durability potential of concrete (Alexander 1997). Such index tests must be sensitive to important material, processing and environmental factors affecting concrete. The purpose of material indexing is to provide a reproducible engineering measure of microstructure and properties important to concrete durability at a relatively early age (e.g. 28 days). Thus it should be possible to produce concretes of similar durability by a number of different routes: additional curing, lower w/c ratio, different binder types, etc.

In order to assess the effectiveness of these laboratory techniques, field studies of concrete in service are essential. These studies allow deterioration mechanisms to be accurately assessed under normal exposure conditions. The disadvantage of field exposure testing is that deterioration may take years to proceed to a measurable extent thereby requiring extrapolations to estimate long-term trends. These extrapolations may be misleading if the results are not independently validated with long-term data.

## 2 LABORATORY TESTING

Transport of aggressive agents into concrete is primarily caused by absorption, permeation and diffusion mechanisms. The influence of each transport type is dependent on environmental conditions and material properties of concrete. Carbonation-induced corrosion of reinforcement is caused by gaseous diffusion of carbon dioxide through relatively dry concrete. Chloride-induced corrosion in contrast is caused by ionic diffusion through near- or fully saturated material.

Results reported in this paper have been produced from several different South African studies over the last ten years. Concrete mixes were produced with local materials and are denoted: PC – 100% Portland cement, FA – 30% fly ash, SL – 50% blast-furnace slag and SF – 10% condensed silica fume. Grade of concrete refers to the nominal concrete cube strength of the material at 28 days.

### 2.1 Oxygen permeability test

Permeability is defined as the capacity of a material to transfer fluids under the action of an externally applied pressure. The permeability of concrete is dependent on the concrete microstructure, the moisture condition of the material and the characteristics of the permeating fluid. Ballim developed a falling-head permeameter that allowed simple measurement of oven-dried concrete exposed to oxygen under pressure (1993). The test equipment and typical measurements are shown schematically in Fig. 1.

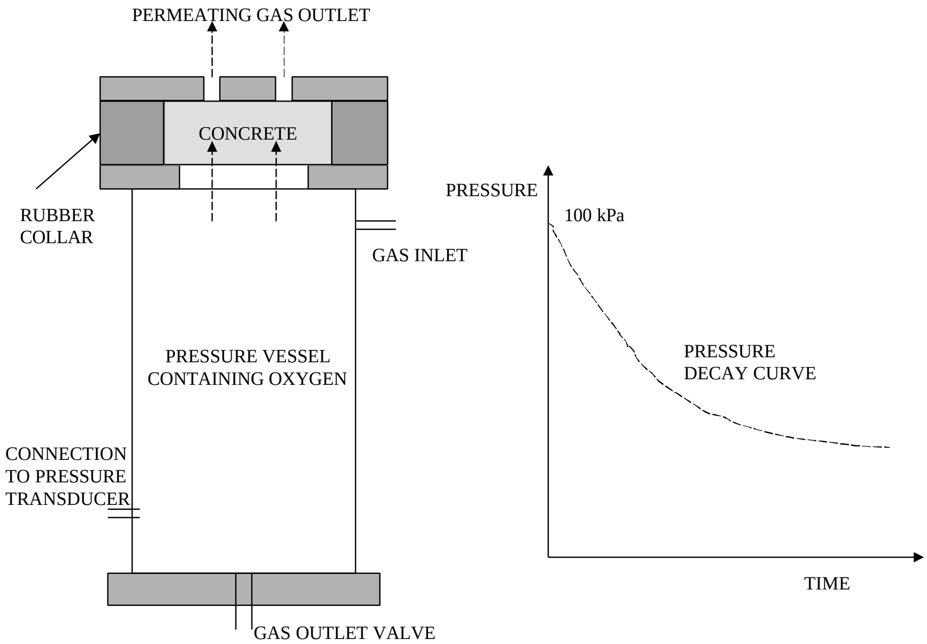


**Figure 1: Oxygen permeability apparatus and typical test data**

Concrete core samples are initially dried at 50 $^0$C before being exposed to oxygen at 100 kPa pressure. From the slope of the pressure decay curve (using a logarithmic transformation), the Darcy coefficient of permeability is determined. The coefficient of permeability is an unwieldy exponential number that is simplified by defining the oxygen permeability index (defined as the negative logarithm of the coefficient of permeability). Concrete with oxygen permeability index values above 10.0 may be considered to have excellent impermeability characteristics whilst values below 9.0 indicate poor impermeability. Figure 2 show typical results measured at 28 days after either fully wet or dry curing conditions. The oxygen permeability index values were determined from five different laboratories around South Africa and numerous different concrete types. The high degree of scatter was partly due to the wide range of aggregates types used around the country.

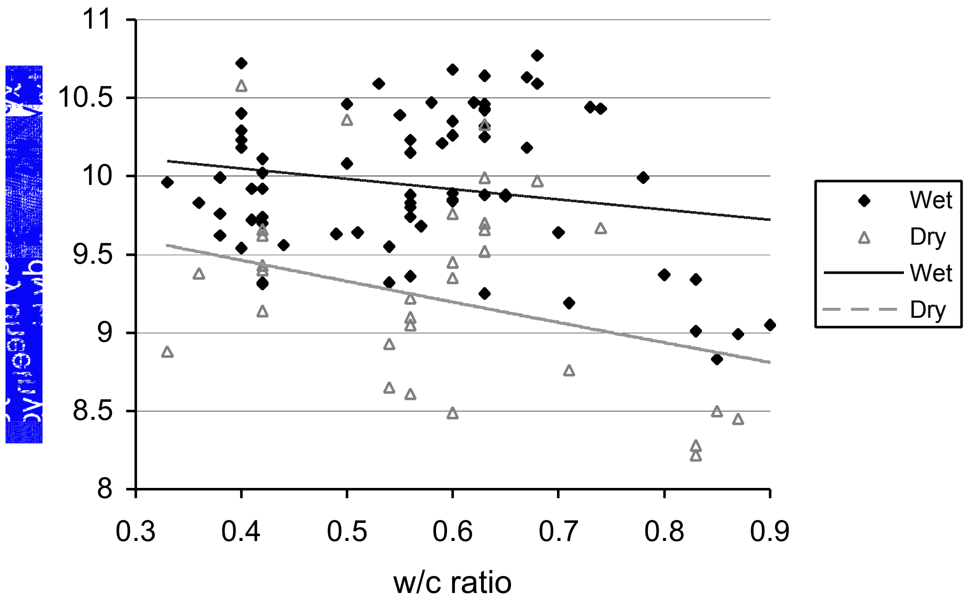


**Figure 2: Oxygen permeability index values at 28 days**

### 2.2 Chloride conductivity test

Diffusion is the process where fluids or ions move through a porous material under the action of a concentration gradient. Diffusion occurs in partially or fully saturated concrete and is the dominant internal transport mechanism for marine concrete. Diffusion rates are dependent on temperature, saturation level of concrete, type of diffusant, chemical interactions and inherent diffusibility of the material. Natural diffusion tests are extremely time consuming and have lead to the development of various accelerated tests.

The chloride conductivity test was developed by Streicher in an attempt to provide a rapid assessment of the chloride resistance of concrete (Streicher & Alexander 1995). Concrete core samples are preconditioned at 28 days to standardize the pore water solution (oven-dried at 50 $^0$C for seven days followed by vacuum saturation in 5M NaCl solution). The sample is then placed in a two-cell conduction rig containing NaCl solution and a 10V potential difference applied. The apparatus (shown in Fig. 3) allows virtually instantaneous readings under controlled laboratory conditions.

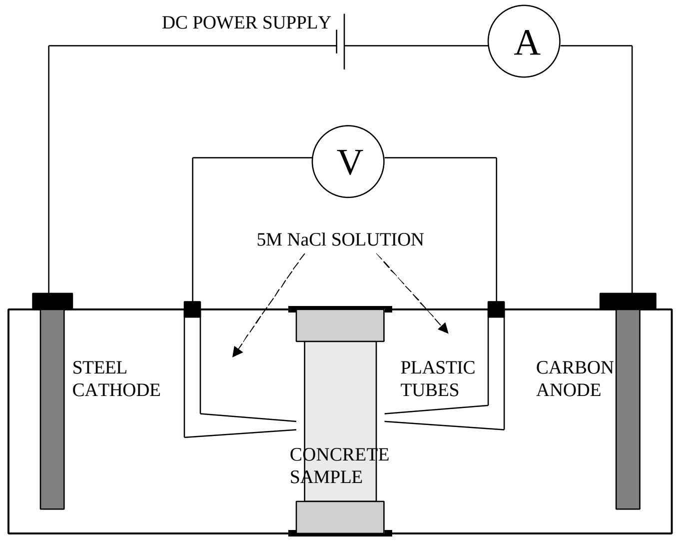


**Figure 3: Chloride conductivity apparatus**

Chloride conductivity testing has been found to be sensitive to changes in concrete microstructure caused by w/c ratio, initial curing and type of binder. The type of cementitious material has a significant effect on chloride conductivity. Portland cement concrete for instance generally has high conductivity values with only high-grade material achieving values below 1.0 mS/cm, which may be defined as excellent chloride resistance. Slag concrete is contrast has significantly lower chloride conductivity values as shown in Fig. 4.

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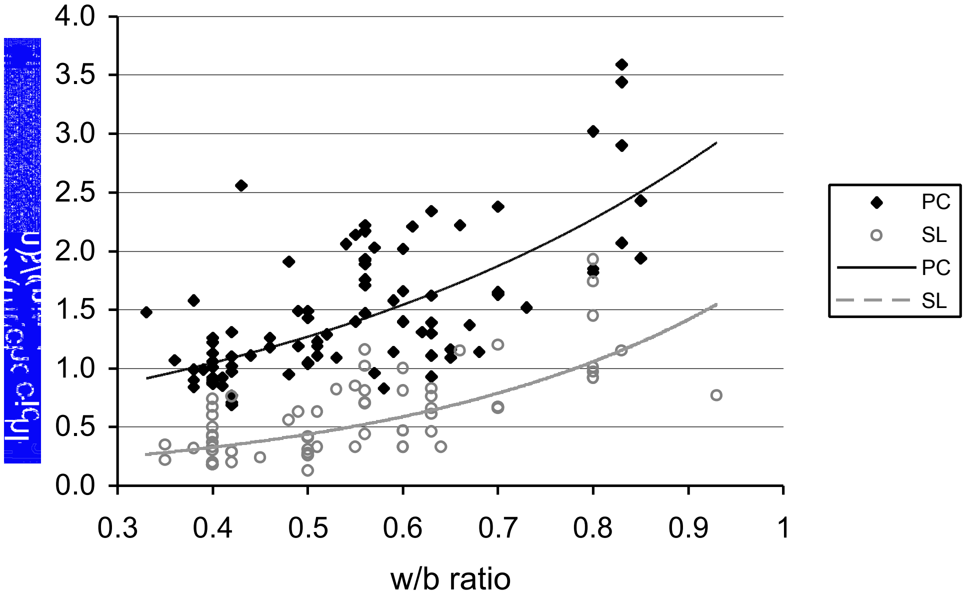


**Figure 4: Chloride conductivity values measured at 28 days (wet cured concrete)**

## 3 DURABILITY PERFORMANCE

Corrosion of steel in concrete is a complex phenomenon, influenced by many internal and external factors. In order to quantify durability performance of concrete, only depassivating effects such as carbonation and chloride ingress are considered. In other words, only the initiation part of the corrosion cycle is considered and factors controlling subsequent corrosion propagation and damage are ignored.

### 3.1 Carbonation

Carbonation of concrete is affected by material, constructional and environmental factors. Effective curing of concrete is known to enhance the near-surface quality of concrete and is particularly important for fly ash and slag concrete. The amount of carbonatable material, in the form of calcium hydroxide, is another important factor. Typical carbonation depths for grade 40 concrete exposed to mild outdoor conditions (average humidity of 80%) are shown in Fig. 5. Concrete samples were large blocks that were cored and tested with phenolphthalein indicator solution. Moist refers to seven days initial moist curing while dry indicates no active moist curing was done.

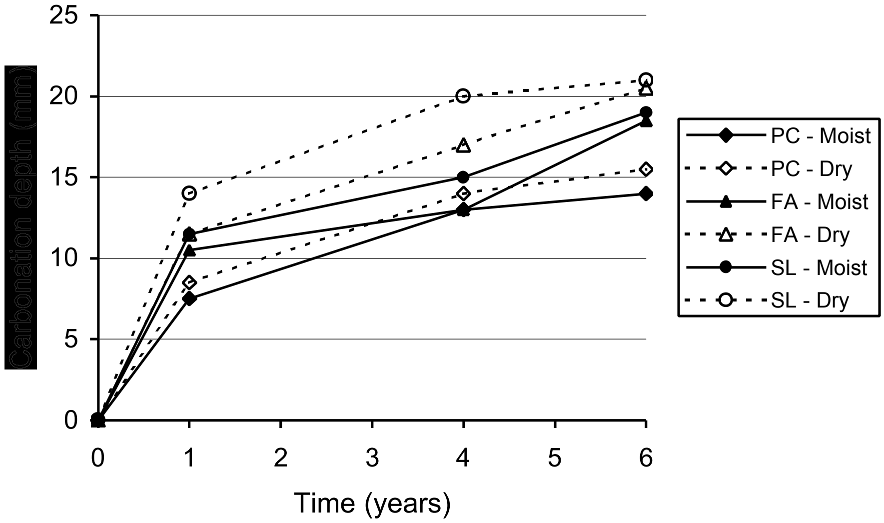


**Figure 5: Carbonation depths for grade 40 concrete (outdoor mild exposure)**

The rate of carbonation is also dependent on environmental conditions during service. A relative humidity of approximately 65% is generally believed to be the optimum condition for carbonation, being dry enough to allow rapid gaseous diffusion of carbon dioxide whilst allowing sufficient moisture for the carbonation reaction to proceed. Table 1 shows carbonation depths for similar concretes exposed to either dry laboratory conditions (temperature 23 $^0$C and R.H. 60%) or mild outdoor environment (average temperature 17 $^0$C and R.H. 80%). Results confirm that drier environments allow significantly higher carbonation depths, particularly when concrete is inadequately cured.

**Table 1: Carbonation depths for concrete exposed to different environments – 4 years**

| Grade (MPa) | Initial Curing | PC 80% | PC 60% | FA 80% | FA 60% | SL 80% | SL 60% |
|---|---|---|---|---|---|---|---|
| 20 | Moist | 13.0 | 18.5 | 13.0 | 21.5 | 15.0 | 23.5 |
| | Dry | 14.0 | 21.0 | 17.0 | 37.0 | 20.0 | 41.0 |
| 40 | Moist | 5.0 | 9.5 | 5.0 | 10.0 | 6.0 | 14.5 |
| | Dry | 6.0 | 12.0 | 6.0 | 13.0 | 9.0 | 24.5 |
| 60 | Moist | 1.0 | 3.0 | 1.5 | 6.0 | 2.0 | 4.5 |
| | Dry | 2.0 | 6.0 | 2.0 | 14.0 | 3.0 | 18.5 |

The rate of carbonation was found to agree with the general power series relationship given in equation 1.

$$x = k_c t^{0.4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where x is the carbonation depth in millimetres, $k_c$ is a material coefficient and t is time in years. The equation allows simple extrapolations of long-term carbonation depths from medium-term data (i.e. one to seven years exposure).

### 3.2 Chloride ingress

Chloride ingress into concrete was assessed by exposing concrete blocks to a range of marine environments. Exposure testing was conducted at two sites on the Cape Peninsula in South Africa with five separate projects being undertaken over periods from one to seven years. Concrete cores were extracted from the concrete after exposure and sliced into fine increments for chloride analysis. Total acid-soluble chloride concentrations were determined in accordance with BS1881 but using a potentiometric titration (British Standards 1988). From the measured chloride profile, the surface concentration and diffusion coefficient were determined using the solution of Fick's second law of diffusion.

Typical results for grade 40 concrete exposed to very severe marine exposure (tidal zone location but with limited wave action and abrasion) are compared. Water/binder ratios were in the region of 0.50 for these concretes (Mackechnie & Alexander 1997a). Surface concentrations at the different ages are shown in Fig. 6. PC concrete was found to have variable levels whereas surface concentrations for FA and SL concrete increased significantly with time although appearing to be stabilizing by eight years. The variable surface concentrations of PC concrete had an impact on measured diffusion coefficients, these two parameters being directly affected by one another in the Fick's law equation.

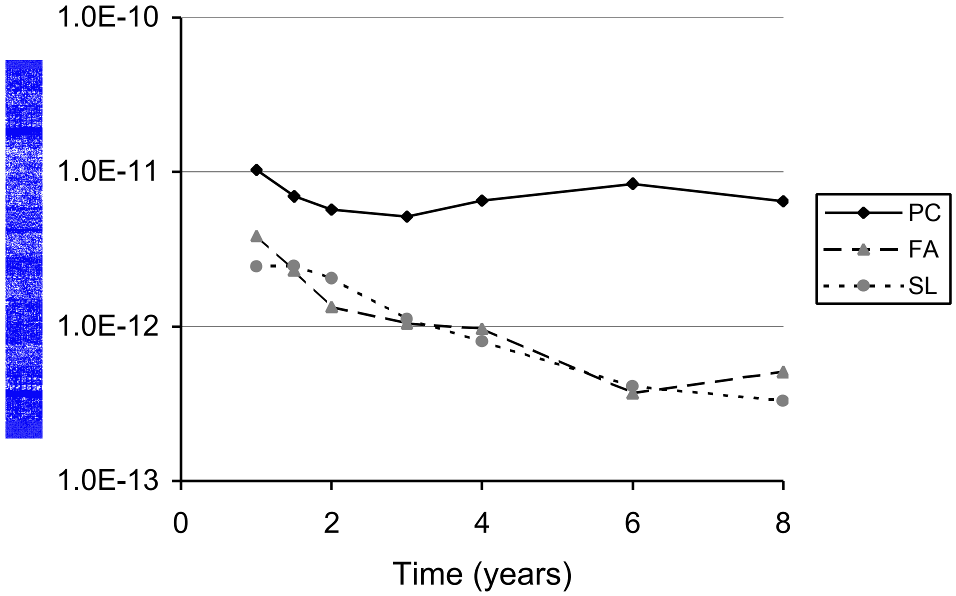


**Figure 6: Surface concentrations with time**

Diffusion coefficients were found to reduce with time, particularly for FA and SL concrete. After eight years exposure, PC concrete was found to have relatively high diffusion coefficients such that reinforcement was at risk of corrosion to depths greater than 70 mm. Figure 7 illustrates the change in diffusion coefficients with time for grade 40 concrete exposed to very severe marine conditions at Simonstown tidal zone.

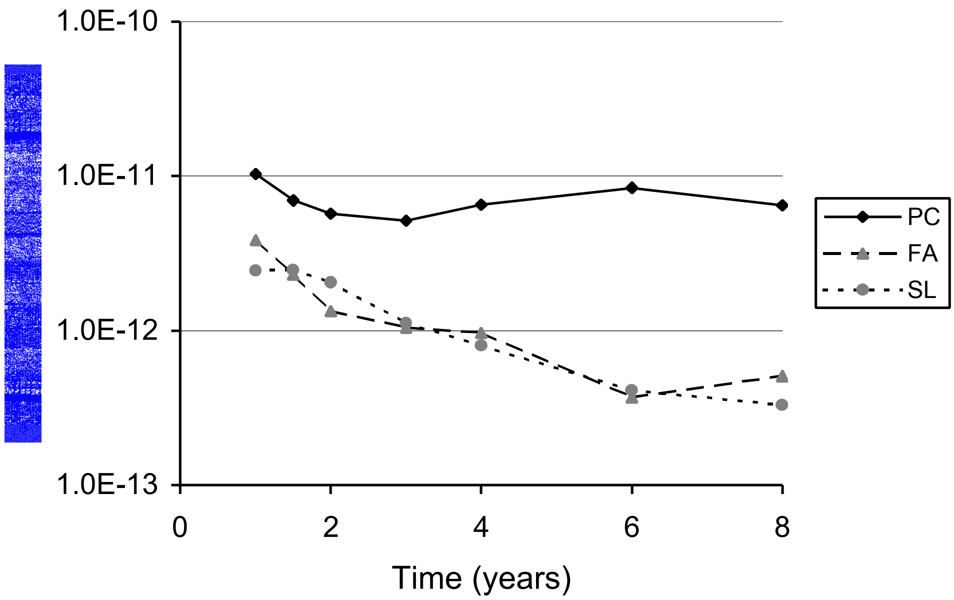


**Figure 7: Diffusion coefficients with time**

Analysis of chloride ingress data revealed what other researchers have observed, namely that Fick's law of diffusion is not able to accurately predict chloride levels in concrete (Bamforth 1994). Some allowance needs to be made for the reduction in diffusion coefficient values with time, particularly for fly ash and slag concrete. A modified solution of Fick's second law of diffusion was therefore formulated based on the empirical relationship given in equation 2 (Mackechnie 1996).

$$D_c = D_i \, t^{-m} \quad \text{..........................} \quad (2)$$

where $D_i$ is the diffusion coefficient at one second, m is the empirical material coefficient relating to the reduction in diffusion coefficient ($D_c$) with time. The modified solution of Fick's second law of diffusion can therefore be expressed as equation 3.

$$C_x = C_s\,(1 - \mathrm{erf}[\frac{x}{2\sqrt{D_i}\, t^{(1-m)}}]) \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where $C_x$ is the chloride concentration at depth x and time t, and $C_s$ is the surface concentration which is assumed to be constant.

## 4 DURABILITY PREDICTIONS

Service life predictions of reinforced concrete structures are affected by numerous variables that prevent precise estimates of durability performance. Since durability index tests are based on transport mechanisms associated with deterioration, it was thought that these indexes could be used for durability predictions. Durability predictions of an empirical nature were therefore sought using correlations between early-age characterization tests and durability performance results.

### 4.1 Carbonation predictions

Correlations between oxygen permeability index values recorded at 28 days and carbonation depths after exposure were found to be good. Figure 8 shows the relationship for concrete after four years exposure. Oxygen permeability testing was found to be more sensitive than strength in predicting the carbonation resistance of concrete.

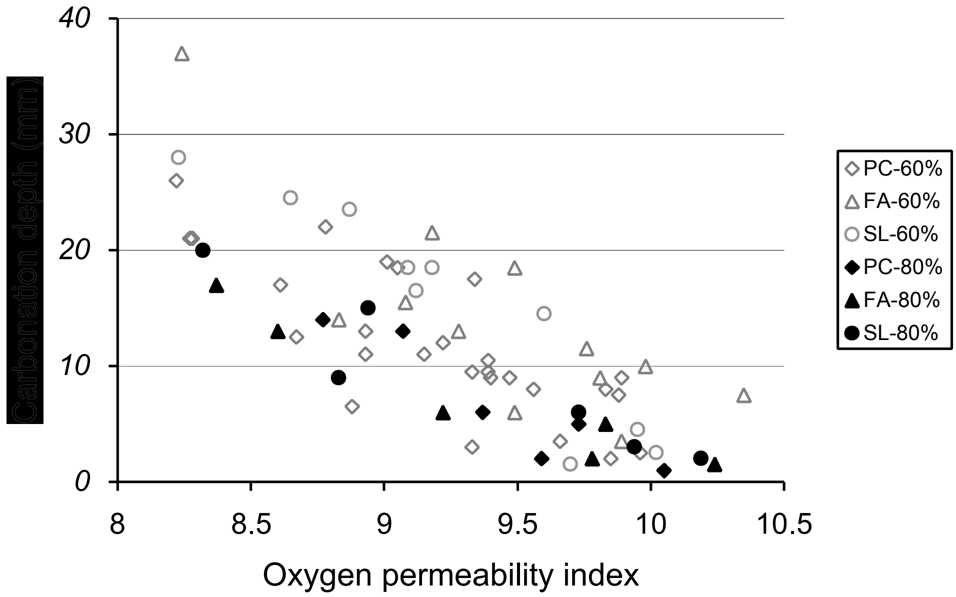


**Figure 8: Carbonation versus oxygen permeability index**

An empirical prediction model for carbonation was therefore formulated using the oxygen permeability test. Early-age characterization testing was used to estimate medium-term carbonation depths, from which long-term results may be extrapolated using equation 1. Using this approach 50 year carbonation depths may be predicted for different environments as shown in Fig. 9

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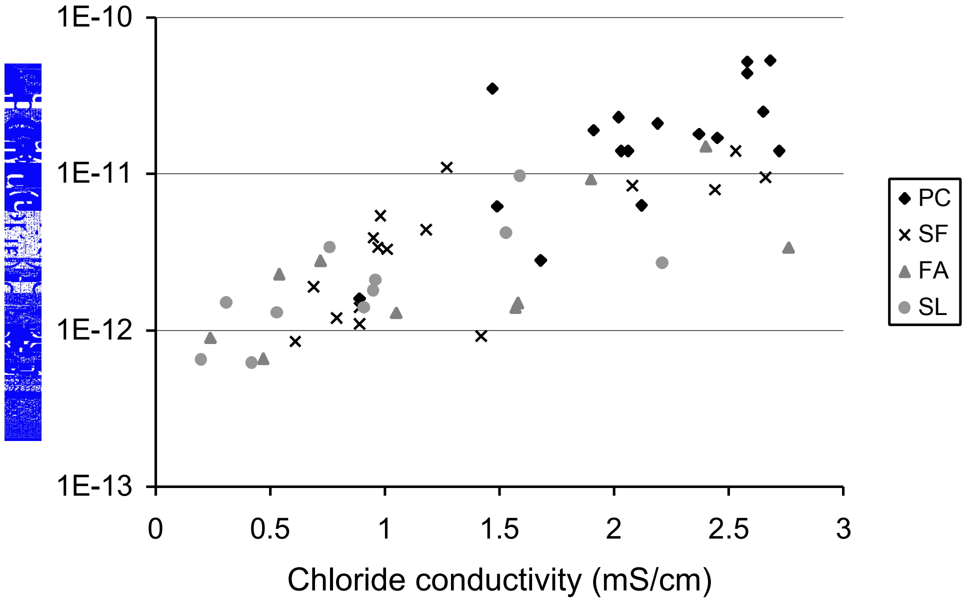


**Figure 9: Predicted carbonation depths after 50 years**

### 4.2 Chloride predictions

Chloride diffusion through concrete is a complex process involving material, environmental and structural effects. Correlations between early-age properties and durability performance are complicated by material-specific responses. This was observed when correlations between diffusion coefficients and 28 day chloride conductivity values were analysed. Figure 10 shows the relationship found for concrete exposed to very severe marine environment for two years. The two year correlation is shown as results from later ages did not include the full data series but showed similar trends.

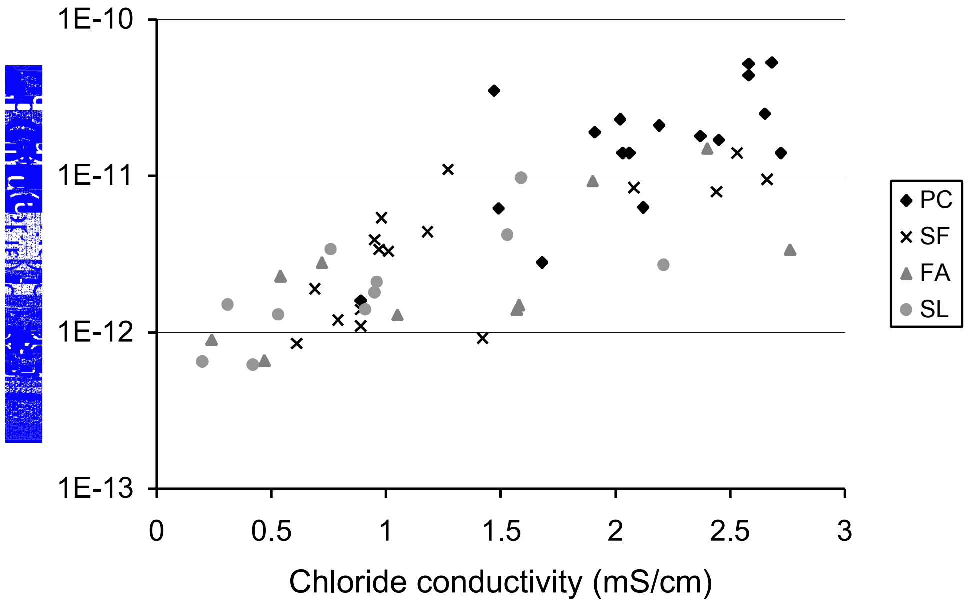


**Figure 10: Diffusion coefficient (two years exposure) versus chloride conductivity**

A more reliable relationship was found when longer-term chloride conductivity values were compared with diffusion coefficients. Some allowance therefore needs to be made for the differing rates of maturity of concrete and chemical interactions between diffusant and concrete. A prediction model based on this premise has been formulated and has been found to have reasonable reliability when independently validated using long-term site data (Mackechnie & Alexander 1997b).

The prediction model consists of using 28-day chloride conductivity testing to predict long-term diffusion coefficients but allows for longer-term cementing reactions and chemical interactions between chloride ions and concrete. From the model, time to corrosion activation may be estimated for different concrete types. Figure 11 shows predictions for moist cured concrete, exposed to very severe marine conditions, with cover to reinforcement of 60 mm (note that silica fume data is somewhat conservative due to limited long-term data, i.e. two year exposure only).

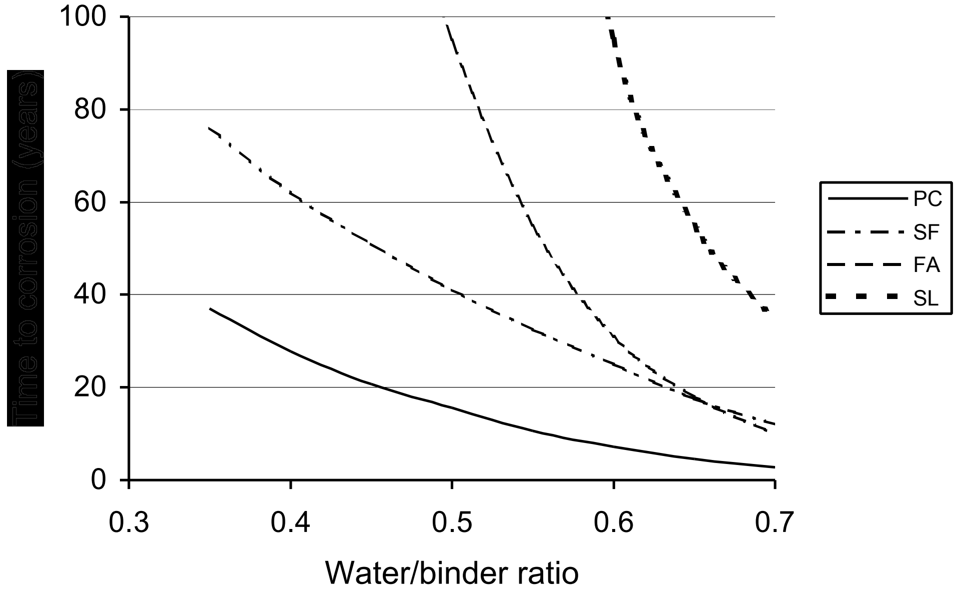


**Figure 11: Time to corrosion for reinforced concrete – very severe exposure**

## 5 CONCLUSIONS

A pragmatic approach is proposed to solve the problem of lack of durability in concrete structures. This approach takes a broad view, incorporating a proper definition of the environment, characterization of the material and field observations of durability performance. By integrating these various aspects, performance specifications can ultimately be produced to achieve specified durability criteria.

Durability index tests such as oxygen permeability and chloride conductivity were found to be sensitive to material, environmental and processing factors that affect concrete durability. The tests were also found to produce reasonable predictions of durability performance under a range of environmental conditions. Further, the durability index approach has a sound theoretical basis while still be sufficiently quick and practical for site use.

## 6 ACKNOWLEDGEMENTS

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# Methods Of Testing Alkali-Silica Reactivity In The Netherlands

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**Summary**: **In the current Dutch standard NEN 5905 and CUR Recommendation 38, four methods are listed for assessing the alkali-silica reactivity of concrete aggregates. However, procedures outlining <u>how</u> the tests should be performed, and the possible errors associated with such tests, are not given. This makes interpretation of results obtained by any of these methods quite difficult and often impossible. In 2000, the Dutch Ministry of Transport, Public Works and Water Management published a draft procedure for assessing the potential alkali-silica reactivity of concrete aggregates in a fast but robust manner. This procedure, which is intended to form one of the backbone's of the currently reviewed CUR Recommendation 38, consists of a combination of petrographic examination and expansion measurements. For expansion measurement, mortar bars are prepared with the test aggregate, following an assessment in an ultra-accelerated mortar-bar test based on the RILEM TC 106-2 method. This article summarizes the essential elements of the new test procedure, and presents results of tests of several primary as well as secondary (recycled) aggregates.**

**Keywords. alkali silica reaction, petrography, microscopy, accelerated mortar bar resting, (recycled) aggregates**

## 1 INTRODUCTION

Alkali-silica reaction in concrete, ASR, is a chemical reaction between the alkalis in the pore solution of the cement paste and certain forms of silica which occur in the aggregate. The product of the reaction is a gel that can imbibe water and swell. In some cases, swelling of the gel can induce internal stresses in the concrete of such magnitude that extensive cracking leading to structural damage and other durability problems can occur. In reinforced concrete structures, corrosion of the reinforcement can occur as a result of the ASR-initiated cracking.

Worldwide, damage of concrete due to alkali-silica reaction continues to increase. In the Netherlands, for example, since the past decade, deterioration of concrete due to attack by alkali-silica reaction has been detected in several structures including bridges, locks, balconies and viaducts. Two example cases are shown in Figure 1. Recent analysis shows that there are at least 60 structures that are affected or suspected to be suffering from deterioration due to alkali-silica reaction (Heijen et al, 1996). The expectation is that this number of affected structures shall increase in the course of time, even though the Dutch widespread and decades long application of CEM III blast furnace slag cements and portland fly ash cements.

The current Dutch standard NEN 5905 (1995) and CUR Recommendation 38 (1994) authorise suppliers of concrete aggregate to provide information regarding their potential alkali-aggregate reactivity. According to these two documents, the potential alkali-aggregate reactivity can be assessed on the basis of one or more of the following:

- long-term experience with the use of the aggregate in concrete
- microscopical analysis of the mineralogical composition of the aggregate
- expansion tests using mortar bars or concrete prisms
- results of a chemical determination of the reactivity.

**Figure 1: ASR-attack in the foundation of a viaduct (left) and a lock (right) in the Netherlands.**

The four methods have just been listed but clear procedures outlining how the tests should be performed and the expected limitations, variations and possible errors have not been given. Thus making it difficult to interpret results obtained by any of these methods. In 2000, the Dutch Ministry of Transport, Public Works and Water Management (Bouwdienst-RWS), published a draft procedure (BSRAP-R-00006, 2000) for assessing the potential alkali-silica reactivity of concrete aggregates. The procedure consists of two methods:

- **PFM**: petrographic examination of aggregates to identify and quantify rock types and minerals which might react with alkali hydroxides from the concrete pore solution to produce expansive alkali-silica gel.
- **UAMBT**: ultra accelerated mortar-bar test, designed to determine rapidly the potential alkali-reactivity of aggregates through evaluation of the expansion of mortar-bars immersed in NaOH solution at a temperature of 80 $^{o}$C.

As a general rule, the PFM-method is the first step in the assessment of potential reactivity of concrete aggregates. In the Bouwdienst-RWS procedure (BSRAP-R-00006, 2000), petrographic analysis is mandatory for assessing the potential reactivity of aggregates. Owing to the fact the PFM-method is not able to take into account all the potentially reactive constituents, especially in cases where the aggregate sample contains opal or sandstone particles, the reactivity of the aggregate is further assessed by means of the UAMBT-method. Soon after the publication of the Bouwdienst-RWS procedure (BSRAP-R-00006, 2000), several studies have been performed, including a a limited round robin survey with the view to evaluating the effectiveness and reliability of the procedure.

Essentially, it was found that the Bouwdienst-RWS procedure (BSRAP-R-00006, 2000) provided a sound basis for assessing the alkali-reactivity of aggregates. However, the results of a initial round-robin survey indicated that formulation of various portions and instructions given in certain cases easily gave rise to differences in interpretation of the procedures, resulting in factor 2-5 differences in the results of the petrograhic analysis and the UAMBT tests. A subsequently organised workshop (Larbi & Haverkort, 2001) catalised the process of reformulating and further explaining and clarifying the steps set out in Bouwdienst-RWS procedure (BSRAP-R-00006, 2000). At present, it has resulted in a draft revised document, which is intended to be published together with the reviewed CUR Recommendation 38, early 2002.

In the latest version, a specification of 2 %, which was proposed as the maximum acceptable content of potentially reactive constituents in the Bouwdienst-RWS procedure (BSRAP-R-00006, 2000), is currently enforced. Problems regarding reproducibility and limitations, especially in the case of impure sandstone particles, which are also known to be potentially reactive, are yet to be resolved. Current experience (Larbi & Haverkort, 2001) shows that not all types of sandstone have been found to be potentially reactive. As such, the sandstone particles are not directly quantified during the PFM-analysis but rather indirectly evaluated in the UAMBT-method. Work is currently going on to assess which particles should be included.

## 2 THE DUTCH *DRAFT* PETROGRAPHIC METHOD

### Principle

The currently proposed Dutch *Draft* Petrographic Method of assessing the potential alkali-silica reactivity of aggregates is based to a large extent on the RILEM TC 106-1 (2000) method. The method is intended to be used to assess all types of primary aggregates, that is, natural sands and gravel, crushed fine and coarse natural aggregates, including “all-in” as well as secondary alternative or recycled aggregates. The petrographic method is mandatory and is the first step in the assessment of the potential reactivity of concrete aggregates, as well as quality control in the course of production.

### Reactive minerals

The rocks and mineral constituents that are considered to be potentially alkali silica reactive are currently essentially porous chert, chalcedony and opal. It should be noted that, apart from porous chert, chalcedony and opal, some types of sandstone, quartzite and other mylonitic rock particles have been found to be potentially reactive from diagnosis of Dutch concrete structures affected by ASR. However, at the moment, no investigation has been performed by means of petrography to distinguish between the reactive and non-reactive rock types, although it is generally known that such a distinction may in principle be done by means of petrography. As for now, all such rock types are considered potentially alkali-silica even though many of the particles in question are found to be inert in thin sections prepared from ASR-affected OPC concrete structures. For this reason the potential reactivity of such particles or constituents is assessed after the petrographic analysis by means of the ultra-accelerated mortar-bar test, UAMBT.

**Procedure**
The procedure involves description of various methods and analyses for assessing the potential alkali-silica reactivity representative aggregate sample by means of petrography for use in concrete. It consists essentially of the following main proce and aspects:

- taking samples from a quarry or a stock pile, including the required sample size for testing in the laboratory according to a specified international standard, EN 932-1
- initial preparation of the samples in the laboratory for examination, which includes homogenising, drying, sieving, splitting by means of quartering, and grinding, EN 932-1
- preparation of thin sections from test specimens according to a specified procedure
- number of thin sections required for a given type of aggregate, for example, crushed coarse natural rock, sand (fraction 0-4) or gravel (fraction 4-16)
- point-counting analysis, with special emphasis on the number of points to be counted for various size fractions
- evaluation of results and report writing.

**Sampling**

The aggregate producer or supplier, the investigating institute or consultant, or a certified body according to the procedures enshrined in EN 932-1, may do the sampling. The quantity of sample taken is dependent on the maximum particle size of that fraction and varies from 50 kg for a maximum particle size of 63 mm to 5 kg for particle size of 4 mm.

**Cursory visual examination**

It is as in RILEM TC 106-1, reported by Jensen and Sibbick (2001) elsewhere.

**Initial sample preparation**

The method of drying, splitting and weighing are all similar to those in RILEM TC 106-1 (Jensen and Sibbick (2001); with respect to **grinding**, the method is a little different from that of RILEM TC 106-1.

For sand 0-4, the sample is first sieved into three sub-fractions: 0-2, 2-4 and > 4 mm. The fraction coarser than 4 mm is then cru to a size fraction smaller than 4 mm, added and eventually homogenised with the fraction 2-4.
For gravel and other coarse aggregate particles, the crushing procedure is more comprehensive than the case recommended in the RILEM TC 106-1 procedure. In the present case, the material is first split into four equal parts by the method of quartering. One-quarter is further sieved into three size fractions: 4-8, 8-12 and 8-16 or 4-8, 8-16 and 16-32 or similar size fractions in the case of different aggregate grading. The various fractions are crushed separately, using a crusher with the opening set to less than 4 mm. The crushed material is mixed and wet-sieved on the 63 μm sieve to remove the fraction finer than 63 μm. Following this, the material is then dried at 105 $^{o}$C to a constant weight and homogenised.

**Thin section preparation**

The current method is the same as in RILEM TC 106-1. Of mandatory importance here is the (proper) use of a fluorescent dye in the impregnation resin and impregnating under vacuum in order to recognise and quantify any porous chert present.

**Number of thin sections**

The number of thin sections to be prepared and analysed by means of polarising and fluorescent microscopy for the various aggregate size fractions are as follows:

- sand, fraction 0-1: 1
- sand, fraction 0-2: 1
- sand, fraction 0-4: 1 from fraction 0-2 and 2 from fraction 2-4;
- sand, fraction 0-4: 1 from fraction 0-2 and 2 from fraction 2-4;
- gravel and other coarse aggregate: 3 from processed fraction

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### Point-counting analysis

Same as in RILEM TC 106-1 [6] except that a minimum of 3000 points in the particles in the thin sections must be counted in order to achieve an acceptable actual error, which from a number of research studies has been found to be less than 0.5 %. This is because the maximum acceptable content of potentially reactive constituents in aggregates is quite low, that is, 2 % by volume.

### Determination of the content of PRC

The total content of PRC in the samples, identified and quantified by means of point-counting analysis is determined and recorded for the various size fractions and for that matter the entire aggregate sample.

### Criteria for assessment

If from the petrographic analysis, an aggregate is found to contain less than 2 % (in practice 2,49 % with a 95% confidentiality limit) of the potentially reactive constituents (PRC), it is termed "*not potentially reactive*" but has to be tested further by means of the ultra-accelerated mortar-bar test. If the aggregate contains more 2 % of PRC, it is considered *potentially reactive* and can only be used in concrete structures exposed to dry environmental conditions. If it is intended to be used in concrete structures exposed to either wet, seawater, frost-thaw and/or aggresive conditions, the necessary preventive measures need to be taken, such as the already in CUR Recommendation 38 prescribed. Examples are the use of a blast furnace slag cement (CEM III/B resp. CEM III/A) containing specified slag contents (# 66 resp. 50 %) and associated $Na_2O$ contents (< 2 resp 1,1 %, still under discussion), or alternatively a OPC containing ∃ 25% PFA and a $Na_2O$ content of < 1,1%.

### Reporting

Same as in RILEM TC 106-1 reported by Jensen and Sibbick [6] elsewhere.

## 3 THE DUTCH *DRAFT* ULTRA-ACCELERATED MORTAR-BAR TEST METHOD

The method used is the same as that outlined in the RILEM TC 106-2 method (2000). Of importance here is the method of sampling, sample homogeneity and preparation, for which an additional clarification is written.

## 4 PILOT TESTING

A summary of some results of a pilot study, aimed at evaluating the limitations and general suitability of the current *draft* Dutch procedure is presented in table 2. Micrographs showing some features of the test results are presented in Figures 2, 3 and 4.

**Table 2: Summary of results of a pilot study using the new draft Dutch procedure**

| *Material* | *Size fraction* | *PRC %* | *Petrographic Assessment* | *UAMBT Expansion, %* |
|---|---|---|---|---|
| River sand | 0 - 1 | 0.5 | Under critical | 0.02 |
| River sand (reference) | 0 - 2 | 1.1 | Under critical | 0.01 |
| River sand | 0 - 4 | 2.8[1] | Potentially Reactive | 0.12 |
| Sea sand | 0 - 4 | 4.5[1] | Potentially Reactive | 0.23 |
| River gravel 1 | 4 - 16 | 1.2 | Under critical | 0.04 |
| River gravel 2 | 4 - 32 | 1.5 | Under critical | 0.03 |
| Crushed siliceous limestone (reference) | 4 - 22 | 2.8[1] | Potentially reactive | 0.26 |
| Recycled concrete | 4 - 32 | 1.8 | Under critical | 0.02 |
| Recycled concrete containing additional 1 % (by mass) of glass[2] | 4 - 32 | 2.7[1] | Potentially reactive | 0.12 |
| Assessment criteria | - | ≤ 2 | - | 0.1 at 14 days |

[1] Under normal circumstances, these samples with more than 2 % potentially reactive constituents would not be tested further using the UAMBT. However, for purposes of comparison and evaluation of the effectiveness of the new procedure, the sample tested further using the UAMBT.

[2] According to NEN 5905 the maximum allowable content of "foreign constituents" is 1% (m/m).

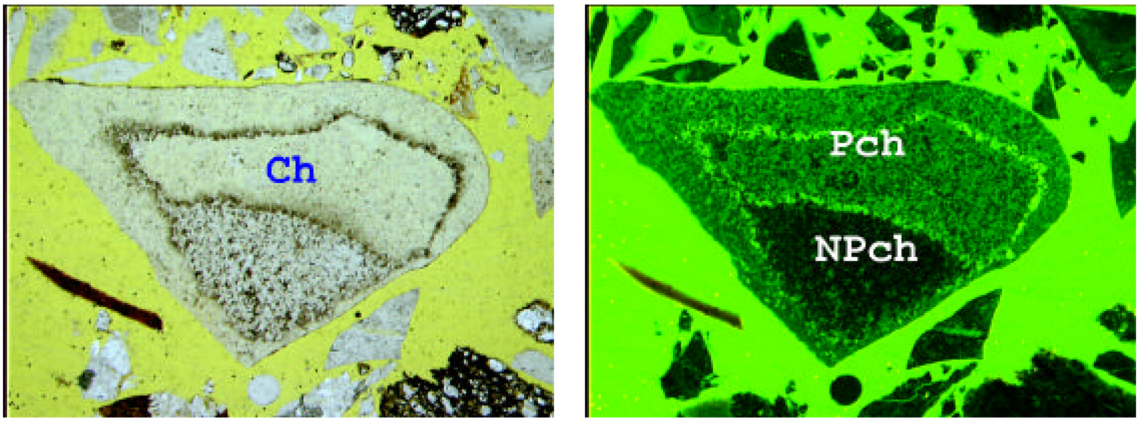


**Figure 2: PFM-micrographs showing porous chert, Pch, in one of the thin section examined using the petrographic method. NPch = non-porous chert. Micrograph is 2.7 mm x 1.8 mm.**

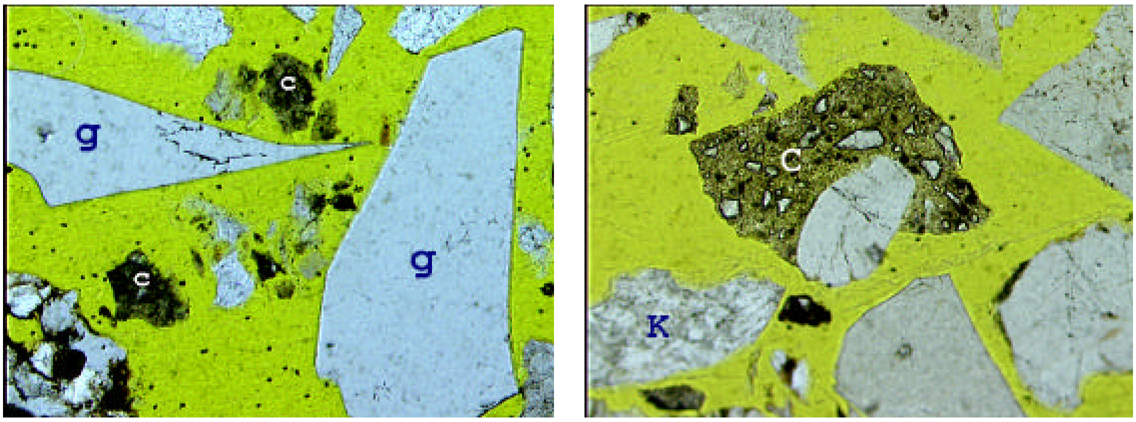


**Figure 3: PFM-micrographs showing a glass particle, g, in one of the thin sections prepared from recycled concrete and examined using the petrographic method. C = cement paste. Micrograph is 2.7 mm x 1.8 mm.**

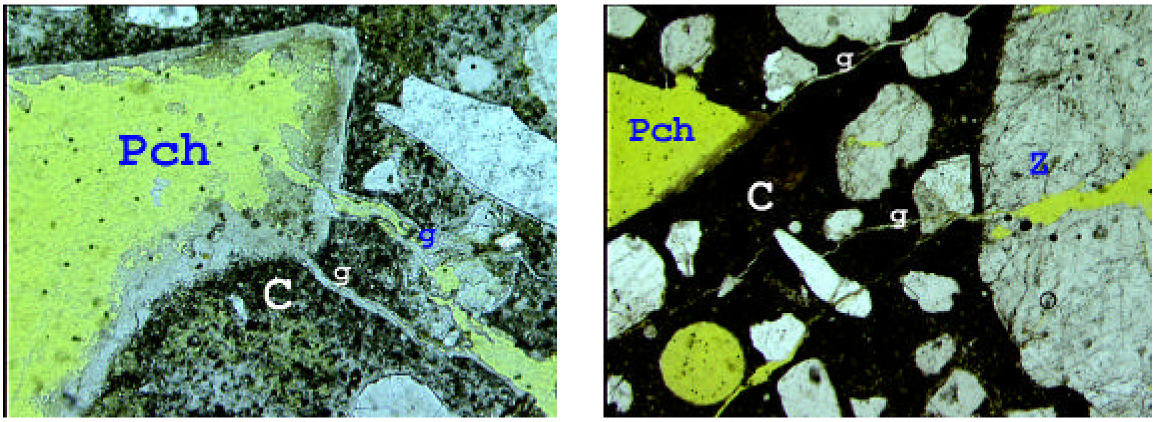


**Figure 4: PFM-micrographs showing ASR-gel streaming from two reacted aggregate particles in the mortar-bar of one of the samples of sand tested. Pch = porous chert; Z = sandstone; C = cement paste; g = ASR-gel. A = aggregate. Micrograph is 2.7 mm x 1.8 mm.**

Although the results of this pilot test are somehow limited, it can be deduced that a confirmation of the results of the UAMBT, including those of the two reference samples indicates that the Draft method has much potential. The data in table 2 further show that for the two sand samples (fractions 0-1 and 0-2) and both samples of river gravel (fractions 4-16 and 4-32), the presence of sandstone, quartzite and other mylonitic rock types present in the samples have little effect on their potential alkali-silica reactivity. A more extensive and detailed round-robin test involving about seven laboratories in The Netherlands, including two recognised laboratories in Belgium and Denmark is scheduled to take place early 2002.

## 5 DISCUSSION AND CONCLUSIONS

A new, relatively fast and robust, procedure for assessing the potential alkali-silica reactivity of aggregates, partly by means of petrography and partly by means of ultra-accelerated mortar-bar expansion, is currently in preparation and almost completed for use by the Dutch concrete and the building and construction industries.

The two methods in the *draft* procedure are both based on the RILEM TC 106 methods, with some small adaptions. The procedure is intended for use in the assessment of the potential alkali-silica reactivity of ***all types of primary aggregates***, including natural sands and gravel, crushed fine and coarse natural aggregates, and secondary alternative or recycled aggregates.

The method of petrography method is mandatory and is the first step in the assessment of the potential reactivity of concrete aggregates, as well as for quality control during aggregate production. It complies with the general scope for aggregate testing as outlined in CEN TC 154.

Although the procedure is relatively new and not widely tested, the ample experience to date indicates that the current *draft* procedure has much potential.

For materials with more than 2% potentially reactive constituents, a significantly more elaborate and long term (> 1 year testing), RILEM CPT test procedure is currently being considered for review and pilot testing.

## 6 ACKNOWLEDGEMENTS

Both authors wish to express their gratitude to all permanent (and temporal) members of CUR VC62 and the Theme Group ASR of the Dutch Ministry of Transport, Public Works, and Water Management. Without their constructive attitude, this paper could not have been written. However, the arguments and discussions communicated here reflect above all the authors opinion, and by no means reflect the Dutch Government policy, nor the shared opinion of all CUR VC62 members.

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# Application Of A Probabilistic Model For The Prediction Of The Decay Due To Salt Crystallisation Of Masonry Buildings Materials

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**Summary**: **An attempt has been made within an EU Contract, to establish the maximum salt content, below which the surface treatments do not fail. Crystallisation tests were carried out on treated and untreated brick and limestone masonry specimens. Salt solutions with two low concentrations of sodium sulphate were inserted in masonry wallettes treated with a water based water repellent and a consolidant.**

**On the basis of the recorded experimental data, a suitable damage parameter describing the material deterioration process has been chosen. The parameter assumed is the loss of surface material at each measurement. The measurements have been made through a laser device along chosen profiles on the masonry surface. The deterioration process could be interpreted as a stochastic process L(t,l), function of time t and damage l, where l is considered a random variable (r.v.) because of experimental evidence. In this way, for different damage levels $\bar{\lambda}$ it is possible allows us to build the fragility curve for each $\bar{\lambda}$. A fragility curve describes the probability of reaching or exceeding a given damage $\bar{\lambda}$ over time.**

**By using this approach the magnitude of the expected damage over time and the occurrence time of it can be predicted. The results will allow for the investigation on the durability of materials with respect to the treatment used and on the decay process of single and composite materials.**

**Keywords. Salt crystallisation, masonry decay process, stochastic processes, fragility curves.**

## 1 INTRODUCTION

Salt decay is one of the most frequent causes of damage to masonry walls in many environments. The presence of moisture in the walls, due to capillary rise and/or rain penetration, is the vehicle through which soluble salts are distributed in the material. Evaporation takes the salts toward the exposed surfaces of the walls; salts crystallising behind the surface cause delamination and crumbling of the masonry components.

Waterproof or consolidation surface treatments cannot be carried out in the presence of salts due to the possible formation of cryptoefflorescence. Nevertheless an attempt has been made within an EU Contract, to establish the maximum salt content, below which the surface treatments do not fail. Crystallisation tests were carried out on treated and untreated brick and stone masonry specimens. Salt solutions with two low concentrations of sodium sulphate were inserted in masonry wallettes treated with a water based water repellent and a consolidant.

On the basis of the recorded experimental data, a suitable damage parameter describing the material deterioration process has been chosen. The parameter assumed is the loss of material from the surface at each measurement. The measurements have been made through a laser device along chosen profiles on the masonry surface. Therefore, the loss of material is quantified as the variation of the profile depth over time.

The high randomness connected with the material characteristics and with the decay in a natural environment suggests to assume the deterioration process L($\lambda$) as a stochastic process of the random variable $\lambda$ (where $\lambda$ is the loss of material at the surface).

The deterioration process could be interpreted as a stochastic process L($t$,$\lambda$), function of the time $t$ and of the damage $\lambda$, where $\lambda$ is considered a random variable (r.v.) because of experimental evidence. However, for a given time $t^*$ the deterioration process can be taken as a function of the r.v. $\lambda$ only; therefore the process can be modelled with a probability density function (p.d.f.) L($t^*$,$\lambda$) depending only by $\lambda$. In order to model it a Log-Normal p.d.f. is chosen. On the other side, we can consider a given significant damage $\bar{\lambda}$ and the variable time needed to exceed it; thus the deterioration process can be treated as a reliability problem. In this way, for different damage levels $\bar{\lambda}$ it is possible to build the *fragility curve* for each $\bar{\lambda}$. A fragility curve describes the probability of reaching or exceeding a given damage $\bar{\lambda}$ over time.

By using this approach the magnitude of the expected damage over time and the occurrence time of it can be predicted. This approach was previously applied to treated and untreated single masonry units (Garavaglia *et al*, 2001) and it will be here presented the application of this model to the mortar/unit systems (brick and stone masonry wallettes). The important information obtained will allow for the investigation on the durability of materials with respect to the treatment used and on the decay process of the single and composite materials.

## 2 EXPERIMENTAL RESULTS

In the last decade it became clear the limit of tests on single materials, when it is necessary to predict the durability of masonry. It was in fact found by the authors already in 1985 [Binda et al. 1985], that the mortar has a large influence on the durability of bricks under salt crystallisation decay and vice versa, according to the porosity characteristics of the two materials, when combined in a wall. Therefore a crystallisation test on wallettes using as ageing agent a soluble salt was suggested and the test, set up by TNO (Delft, NL), was included in the RILEM TC127MS Recommendations in [RILEM MS. A.1].

Some masonry wallettes (250x200x120 mm) were put with their back side in contact with a salt solution at a chosen concentration and then stored over a layer of dry gravel in a plastic container (open at the top). The upper face of the wallette was exposed to the environment (controlled laboratory environment of 20°C and 50% R.H.). Demineralized water is added after three months, when approximately the wallettes are approaching the constant mass in order to start a new cycle and to accelerate the damage.

Crystallisation tests were carried out on one type of softmud contemporary brick used for restoration and on one natural building stones: a limestone from Noto (Sicily). All wallettes were realised with bedding joints, 15 mm high, based on putty lime. Within the EU contract, it was assumed that the salt concentration depends on the capillary moisture content (CapMC) which is calculated on the basis of the water absorbed by capillary rise in 48 hours. In each specimen an amount of salt solution was introduced, according to RILEM MS.A.1, with different salt concentrations for each type of salt: 1% and 2.5% of Cap.MC, and, referred to the % of weight of the dry specimen, (See table 1). These percentages are the lowest salt concentrations used previously for the single units, chosen also for the wallettes in order to verify whether or not these values can still be considered as a threshold value for brick (stone)/mortar composite. $Na_2SO_4$ was used for all the type of masonry specimens. Before inserting the salt solution, some of the wallettes were treated with one water based water repellent and some with one consolidant, immersing the upper surface into the treatment for 10 sec for the water repellent and 30 sec for the consolidant. One wallette was left blank for each material to be used as a reference for the decay.

The water repellent is a solventless silicone microemulsion concentrate based on silanes and siloxanes, diluted with water to yield microemulsion. The consolidant mainly consists of reactive silicic acid ethyl ester compounds. The material was especially developed to strengthen and consolidate weathered natural stone as well as terracotta, brick, stucco, frescos and loam. The choice of these treatments is suggested by their large use.

**Table 1. Salt solution inserted in the wallettes**

| | *% salt referred to CapMC* | | | | | |
|---|---|---|---|---|---|---|
| | 1% | | | 2.5% | | |
| | *w% of the dry specimen* | | | | | |
| *Treatment* | Consolidant | Water Repel. | Reference | Consolidant | Water Repel. | Reference |
| Softmud Brick | 0.23 | 0.22 | 0.24 | 0.57 | 0.55 | 0.59 |
| Noto stone | 0.15 | 0.16 | 0.16 | 0.37 | 0.38 | 0.40 |

### 2.1 The quantification of the damage

In an aggressive environment one of the most important causes of deterioration for the masonry is the salt crystallization. In (Binda *et al.* 1999) the laboratory test simulating this phenomenon is discussed.

The salt crystallization produces high stresses inside the material; the effect is a continuous crumbling and delamination of the exterior surface of the wall while the inside is left unaltered. For this reason the variation in roughness of the surface has been assumed as measure of the damage occurred to the masonry. A laser profilometer was used to monitor the damage (Fig.1) (Binda *et al.* 1999).

The device allows to draw plots of the wall profile in the chosen positions. Subsequent measurements show how the profile is changing in time due to any superficial decay. In this way it is possible to measure the material loss along the time. The Fig. 2 shows an example of profile for five different measurements. The presence of swelling phenomena compromises the damage

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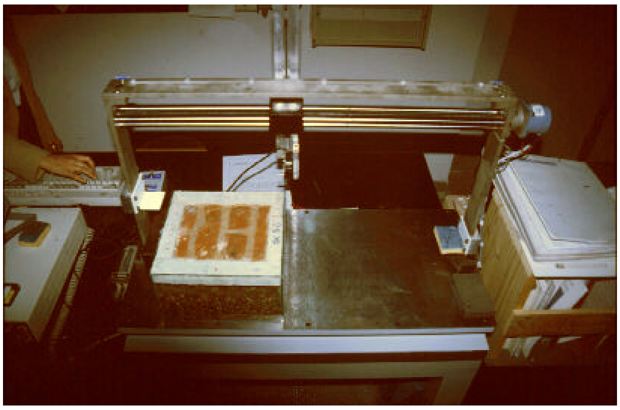
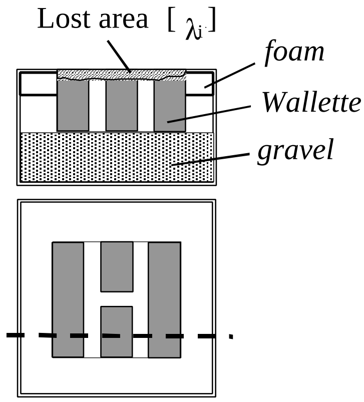
measurements (Fig. 3a). Since bulging is a previous step before detachment, it is possible to consider it as the starting point of a damage. Therefore, through a simple model the experimental measurements have been converted in new deterioration diagrams where the bulging has been considered as a layer loss (Fig. 3b) (Garavaglia *et al.* 2000).

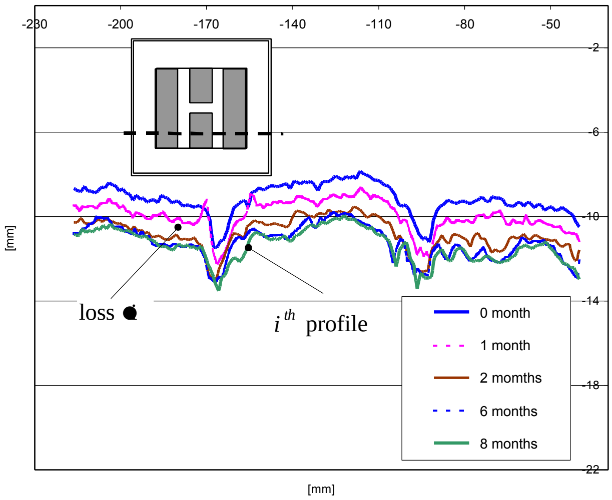


a)




b)

**Figure 1. a) Laser profilometer device during measurement and b) the box scheme**




The loss of material seems to be a good parameter to quantify the masonry damage due to salt crystallisation and can be quantified using the new diagrams of Fig.3b. Therefore, for each profile $i$, represented in Fig. 3a, the loss $\lambda_i$ of cross section of the wall (in $mm^2$), calculated at every time $t^*$ of measurement ($t^*$= 1, 2, 6, 8 months), has been assumed as parameter of damage for the decay due to salt crystallisation. At every time $t^*$, to quantify $\lambda_i$, the area included between two consecutive diagrams is assumed. This area is automatically calculated by the computer code studied to eliminate bulging (Garavaglia *et al.* 2000). To compare all the results obtained, the damage has been plotted in percentage:

**Figure 2. Example of the first measurements realised with laser profilographer on one wallette**

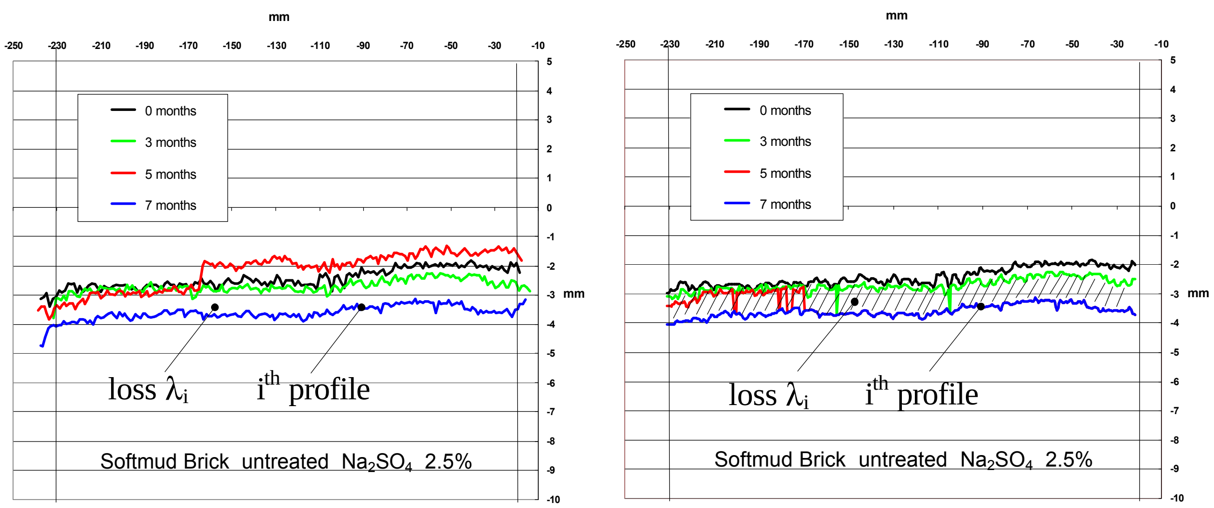



**Figure 3a, b. Example of deterioration measurements over time a) before and b) after the swelling has been removed on a unit of the masonry specimen.**

$$p = \frac{\text{area lost}}{\text{area of the section}} * 100 \qquad (1)$$

A simple interpolation of the experimental point permits to better read the behaviour of the loss $\lambda_i$ over time (linear splices) (Fig.4).

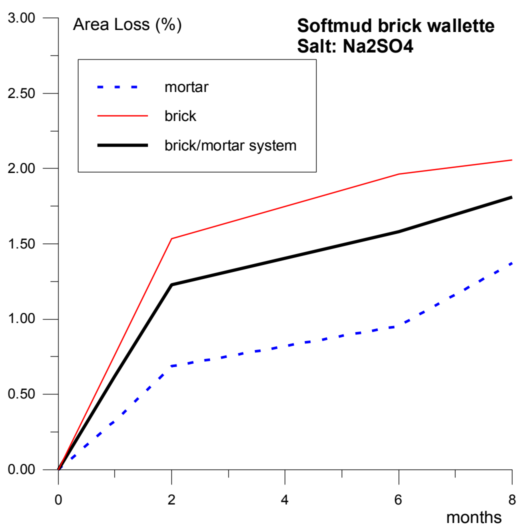



**Figure 4. Deterioration vs. time: example of damage on an untreated brick-wallette**

## 3 THE DETERIORATION PROCESS AS A STOCHASTIC PROCESS IN THE RANDOM VARIABLE l

The deterioration process has been interpreted as a stochastic process function of time $t$ and damage $\lambda$, where $\lambda$ is considered a r.v. because of experimental evidence. However for given time $t^*$ the deterioration process can be seen as function of the r.v. $\lambda$ only**.** The probability density function (p.d.f.) L($t^*$,$\lambda$), describing the behaviour of $\lambda$ at the time $t^*$, can be modelled as a Log-Normal p.d.f. (Fig. 5) (Binda *et al.* 1999, Garavaglia *et al.* 2000). On the other hand, it is possible to consider a given significant damage $\bar{\lambda}$ and the variable time needed to exceed it; thus the deterioration process can be treated as a reliability problem.

Indeed (e.g. Evans 1992) reliability $R(t)$ concerns the performance of a system over time and it is defined as the probability that the system does not fail by time $t$. Here this definition is extended by denoting by $\overline{R}(t)$ the probability that a system exceeds a given significant damage threshold $\bar{\lambda}$ by time $t$. The random variable that is used to quantify the reliability is $\bar{T}$ which is just the time to exceed the damage $\bar{\lambda}$. Thus, the reliability function is given by (Garavaglia *et al.* 2001*a* Garavaglia *et al.* 2001*b*):

$$\bar{R}(t) = \Pr(\bar{T} > t) = 1 - F_{\bar{T}}(t) \tag{2}$$

where $F_{\bar{T}}(t)$ is the distribution function for $\bar{T}$ .

Computing $F_{\bar{T}}(t)$ for different damage levels $\bar{\lambda}$ allows to build up the *fragility curve* for each $\bar{\lambda}$.

A fragility curve describes the probability of reaching or exceeding a given damage $\bar{\lambda}$ over time (Singhal *at al.* 1996). For a chosen damage level $\bar{\lambda}$ at a given time *t**, the probability to reach $\bar{\lambda}$ can be seen as the area under the threshold $\bar{\lambda}$ and the probability of exceeding it can be seen as the area over the threshold $\bar{\lambda}$ (Fig. 6). Indeed, the computed areas over different thresholds $\bar{\lambda}$ provide the experimental data used to fit the fragility curves (Fig. 7). Therefore, the evaluation for different *t** of the exceeding probability, connected with each damage level $\bar{\lambda}$, leads to obtain an experimental fragility curve for each chosen $\bar{\lambda}$.

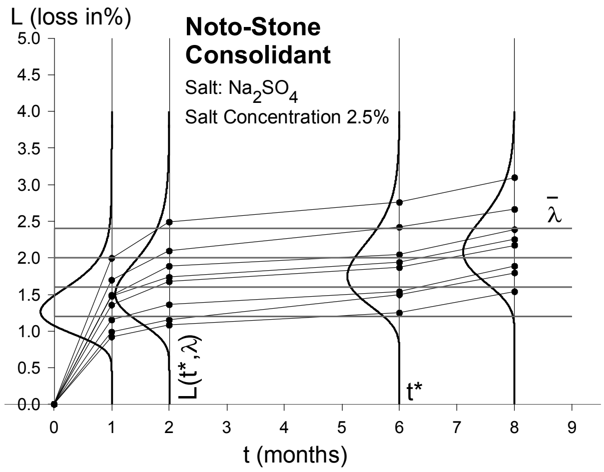



**Figure 5. Interpolation of the loss diagrams (*) and the modelling of the deterioration process L(t*,l)**

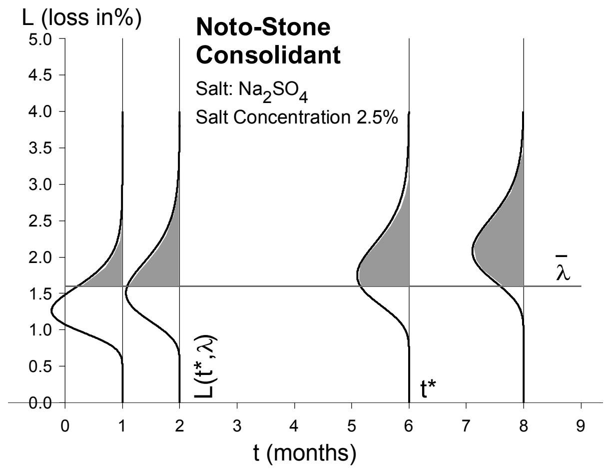



**Figure 6. Exceeding probability to cross the threshold $\bar{\lambda}$**

In order to model the experimental fragility curves, a Weibull distribution has been chosen (Molina *et al.* 1996, Cranmer *et al.* 1998, Bekker 1999, Garavaglia *et al.* 2000). In fact this distribution seems to be a good interpretation of the physical phenomenon (Garavaglia *et al.* 2000).

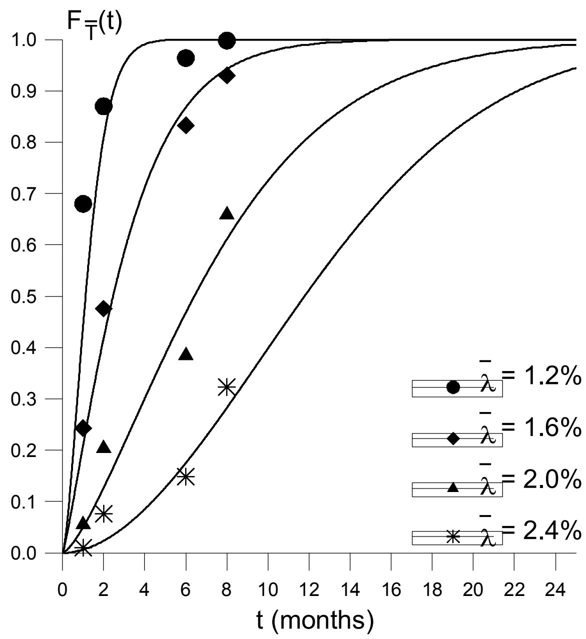



**Figure 7. Fragility curves for different $\bar{\lambda}$ (Noto Stone, Consolidant, Salt $Na_2SO_4$, salts concentration 2.5%)**

## 4 RESULTS AND COMMENTS

### 4.1 Comments on the procedure

The probabilistic approach proposed is able to model the deterioration in terms of probability to reach or exceed a given damage threshold $\bar{\lambda}$ over time. The assumption of the Log-Normal distributions to model the experimental data has pointed out that the deterioration can change its behaviour over time with an increasing scattering. This behaviour is probably due to the randomness connected with the realisation of the decay process (very similar to the deterioration process that happens in the real environment) and to the characteristic of the wallette (i.e.: presence of mortar joints).

Modelling has been made by a computer code involving the maximum likelihood method.

The fitting is done by a computer code involving the least square method. Also in this case the values given by the least square method and the values of the other statistical test performed, associated to the physical knowledge on the deterioration and the statistical knowledge, support the choice assumed.

In each analysed case the fitting of the experimental fragility curve is satisfactory; nevertheless a small number of samples was used. Therefore, in order to interpret these results much caution is needed.

In fact, the number of samples cannot be less than 4 and the time interval must be long enough. A time test too short gives information that can be modelled only by the inferior tail of distribution. As a consequence, the distribution parameters are evaluated on the base of these data, therefore, the fitting can suffer from unreliability.

Of course, this depends also on the investigated damage. If the damage to be consider is small ($\bar{\lambda}$< 0.2-0.4%), the time to reach it is short, therefore the time test during which 4-5 measurements are done can be short (4-5 months). If the damage investigated is serious ($\bar{\lambda}$> 2.0-3.0%) the time test must be longer (10-12 months). In this case it will be possible to make prevision of the damage evolution for a long period of time (more than 30 months). In conclusion the application of this approach is simple, but in order to have significant results the time of monitoring could be very long and the great number of data has to be recorded.

### 4.2 Softmud brick wallettes

The fragility curves of the brick/mortar wallettes with the highest $Na_2SO_4$ concentration are reported in Fig.8, where a comparison of the two treatments with the untreated reference is made. The consolidant determined soon cryptoefflorescences starting from the interface brick/mortar; after 6 months spalling of layers (corresponding to a level of damage of about 1.2%) was still continuing (Fig.8a). On the contrary the water repellent did not produce on the bricks any particular damage within a time of 8 months, therefore the fragility curves represented based on measurements could be referred only to a low level of damage. Since the mortar is the easiest vehicle to water evaporation, the mortar joints are the most damaged, and consequently the decay of the bricks can be expected within two or three years (Fig.8b).

The damage of the reference untreated wallette for the lowest concentration (1%) was very poor in the first three months, than it started uniformly in the bricks and in the mortar joints. With the highest concentration (2.5 %) the damage became soon serious due to cryptoefflorescences on both materials (Fig. 8c).

Comment: as the material loss in untreated wallettes after the same period of time is similar to the ones treated with consolidant, this treatment with this type of bricks seems to be unnecessary to prevent salt crystallisation decay. Up to now no damage is visible on the bricks treated with the water repellent, but the passed experiences suggest to continue the test.

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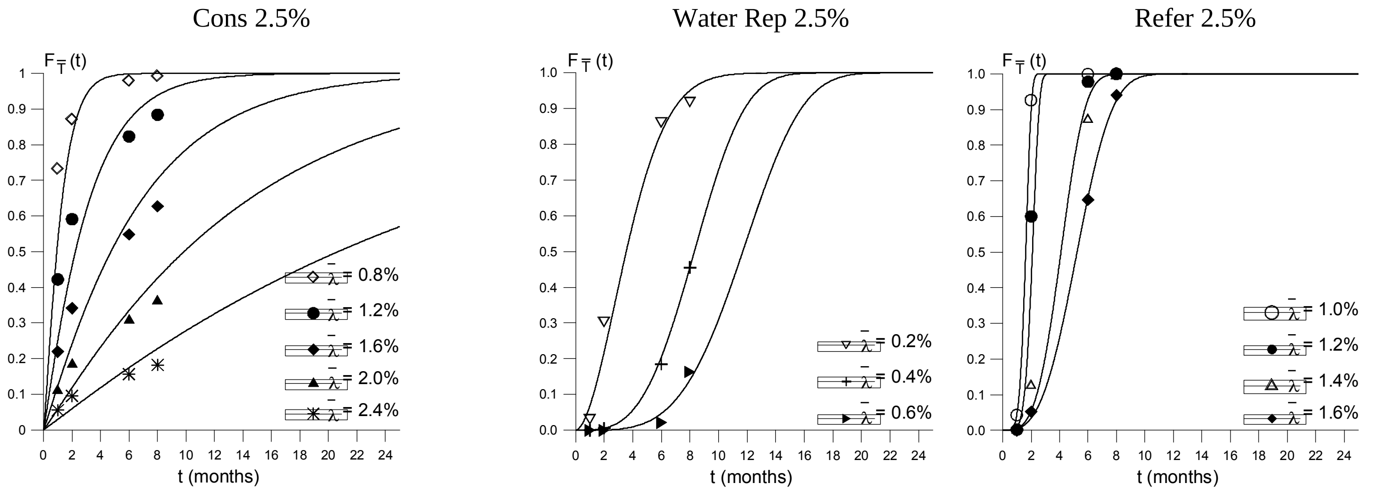



**a)** **b)** **c)**

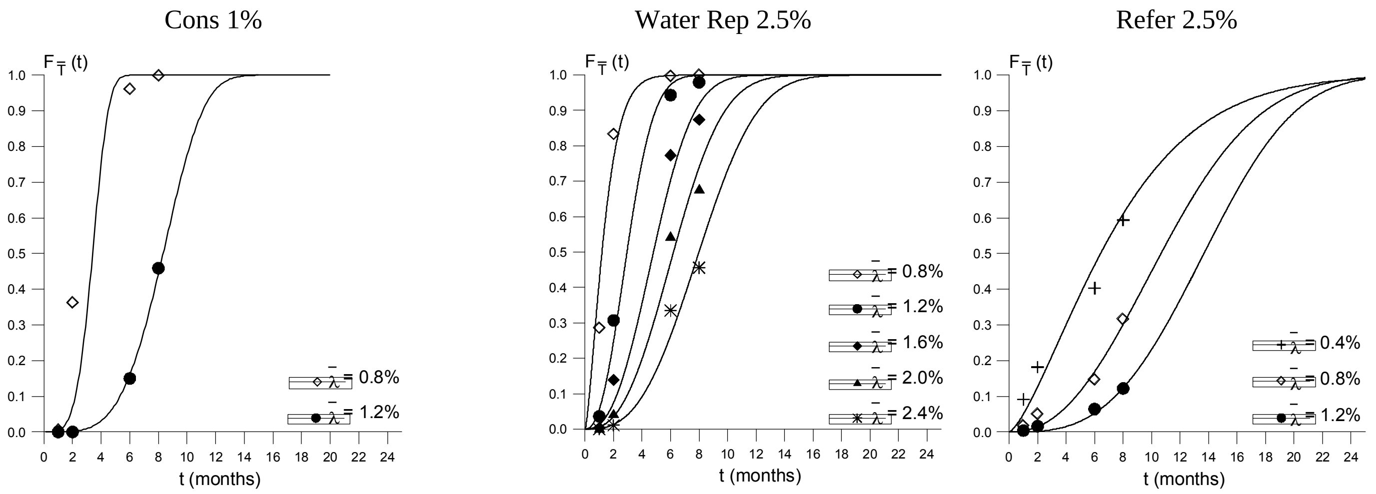
**Figure 8 – Fragility curves built on brick/mortar masonry specimens with different surface treatments.**

### 4.3 Noto limestone wallettes

In the short period of 1 month, the stones treated with consolidant, even with the lower salt concentration, presented detachment of a thin layer (about 0,65 mm), corresponding to a level of damage of about 0.8% (Fig.8a). After the removal of these layers the stones start powdering uniformly. The mortar showed damage after 3 months but only with the higher concentration.

In the case of water repellent, with the lower salt concentration of 1% the damage starts slowly and no particular surface decay is visible on wallettes after the 3 first months (Fig. 9b). After 6 months the damage becomes serious showing exfoliation and spalling of the stones. With the higher salt concentration of 2.5% the damage is serious from the beginning showing spalling of layers of about 1,4 mm (corresponding to a level of damage of 1.2%). After 6 months the observed damage for both concentration is similar (Fig. 9c). Also mortar joints show damage with the two concentrations. 1% of $Na_2SO_4$ could be a threshold but the prevision shows that the probability to reach a serious damage is in two years, as for the reference untreated wallette.

Noto stone, although it is less porous, shows damage before the softmud brick.




a) b) c)

**Figure 9. – Fragility curves built on Noto limestone/mortar masonry specimens with different surface treatments.**

## 5 CONCLUSIONS

During their service life, masonries subjected to an aggressive environment may suffer degradation. The great randomness connected with the occurrence of critical attacks by salt crystallisation suggests approaching the deterioration process of these materials from a probabilistic point of view. The material deterioration process has been approached as a reliability problem

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where the reliability function $R_{\bar{T}}$ has been defined as a function of the r.v. λ = loss of surface material. The failure process is seen as the probability of the system to exceed a given damage level $\bar{\lambda}$. The fragility curves obtained allow to define the exceedance probability over time connected with each $\bar{\lambda}$ chosen.

The approach has been applied on single masonry units and on the same units combined with mortar.

The application of this approach has shown how it is able to predict, in probabilistic terms, the magnitude of the expected damage over time and the occurrence time for a given damage level. As shown, the use of this approach allows to evaluate the treatment effectiveness on different building material.

It has been pointed out how the success of the approach depends on the type of the studied problem and how it is important to have the knowledge of the physical aspects of the analysed phenomenon, in order to correctly model it. It has been shown how the use of this approach is simple, but in order to have significant results, has been underlined that the time of monitoring and the data recorded must be very long.

## 6 ACKNOWLEDGMENTS

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# Developing Rehab Strategies For Drinking Water Networks

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**Summary**: **Urban drinking water networks come at age and need more and more rehabilitation. These needs are largely determined by the length of pipes which have been laid into the ground during past decades with different materials and technologies. There is empirical evidence from failure and rehabilitation statistics that particular pipe types have specific service lives. Thus service life distributions can be applied in a cohort survival model for a differentiated annual forecast of the mileage of pipes reaching the end of their service life and, therefore, being in need of rehabilitation in a particular year.**

**Within the bands of future rehabilitation needs, medium range programs can be designed defining annual targets for specific pipes to be rehabilitated with new materials and technologies. The cost of such rehab programs must be evaluated with respect to their long term effects. Whereas annual rehab investments are derived from specific unit costs, cost savings and other benefits from rehabilitation, during the program period and beyond, are more difficult to forecast. The rehab program may have insufficient effects with respect to reducing failures and leakage and enhancing network service reliability. Some of these effects can be expressed in monetary terms and evaluated with dynamic investment methods in a cost benefit framework.**

**This paper presents a framework for exploring network rehab strategies and describes the method for forecasting the effects of specific rehab programs. A cohort survival model with specific aging functions is linked to a simulation model which calculates the effects of advanced or postponed rehabilitation on some network performance indicators such as failure and leakage rate and average residual life expectancy of pipes in the network. Based on these results, a multi-criteria evaluation procedure is presented for choosing the best rehab program from a set of alternatives. The case of an East German water utility shows how this general approach has been applied for developing a medium term rehab strategy for a the network of water mains.**

**Keywords. Drinking water network, rehabilitation, service life of pipe types, strategic investment planning**

## 1 INTRODUCTION -NEED FOR LONGER VIEW ON INFRASTRUCTURE REHABILITATION

Most water utilities have not developed a long-term rehabilitation strategy, nor do they systematically explore their options for maintaining or upgrading the water distribution network. Usually they decide on a year-to-year base which elements of the water supply system should be rehabilitated. At best, a list of most urgent rehabilitation projects is established and work proceeds along this list of projects that are “in the pipeline” as long as there are funds available and the budget is not cut by other investment needs such as for the supply of new building developments or the repair of unforeseen pipeline damage.

This procedure allows flexible response to whatever comes up, and to some degree, of course, water utilities will always use this re-active “fire brigade” approach, not only because of our limited forecasting capabilities. However, there is a large potential for reducing this “muddling through” and for improving the efficiency of water network rehabilitation. Network information systems, which have been installed in all major water utilities by now, provide a rich source of information which should be used in a pro-active approach to network rehabilitation. Research is under way on this subject (Saegrov et al. 1999, Baur,Herz 1999) in Europe, for example within the CARE-W project, in North America in projects sponsored by the AWWA Research Foundation, and at NRC CNRC (Rajani,Kleiner 2001), and in Australia at CSIRO (Burn 1998) and the home of the Nessie model.

There are bottom up and top down approaches to rehabilitation planning, bottom up starting from individual pipelines, top down starting from the network level. Both approaches need research on the aging behaviour of pipes. Failure forecasting and decision making on whether to go on with spot repair or to do major rehabilitation work, should account for the characteristics of individual pipelines, whereas budget forecasting for maintenance and rehabilitation investments may be sufficiently based on the average aging behaviour of types of pipelines in the network. The network level, in general, seems to be more appropriate for long-term asset management decisions, whereas operational short-term decisions should be based on more detailed information.

Due to our limited forecasting capabilities, some features of the water supply system must be analysed post factum. Water quality for example is monitored and controlled by regular sampling to guarantee that standards are fulfilled. Network performance indicators such as the frequency of bursts, service interruptions, head loss and pressure deficiencies and leakage can be quantified from annual operational statistics and forecast in the short run at least. They indicate whether specific targets of network performance, set by the water utility, are met or need further efforts. Once these targets being set, the utility management is observing how the indicators change and, with some notion on the effectiveness of certain measures, steering the system towards these targets.

The longevity of infrastructure requires a longer view in setting targets of network performance and in developing network rehabilitation strategies. Even the decision of replacing an old pipeline now or postponing its rehabilitation for another year, has to take a long view: At what rate will bursts and leakage increase, what damage will they cause, how long will the new pipeline be in service after rehabilitating the old one? Investments for network improvement need economic justification in a comprehensive cost-benefit framework covering the whole service life of the pipeline.

The result of such an economic analysis depends very much on the full costs of replacement or renovation of pipelines on the one side, and on all direct and indirect benefits resulting from reduced leakage and failures on the other side. These benefits are a function of the water tariff, the damage caused by pipe failures and the repair costs. While investment costs accrue at the beginning and are evenly distributed over a defined financing period, benefits from reduced leakage and repair extend over a longer period and tend to diminish towards the end of the pipe’s service life.

As the benefits from rehabilitation accrue over the whole life of a pipeline, rehabilitation strategies should be explored in the long range, even if they are implemented only in the short or medium range. There is a wide variety of options for doing more or less rehabilitation sooner or later, in a more or less costly way, providing shorter or longer service lives, and there is always the do-nothing alternative: no investments into renovation or replacement, just spot repair of burst pipes.

In the following a framework is presented for exploring rehabilitation strategies with respect to their long range effects. This framework has been applied to many water and gas distribution networks in Europe and USA. It is based on a cohort survival model for infrastructure networks (Herz 1996), which will be briefly presented thereafter. Further methodological details and results will be given in the subsequent illustrative example produced with KANEW, a software developed in a AWWA-RF project (Deb et al.1998) and being further advanced in CARE-W.

## 2 FRAMEWORK FOR EXPLORATION OF WATER NETWORK REHAB STRATEGIES

Future rehabilitation needs are largely determined by the present stock of pipes in the network that have been laid in specific periods with particular materials and technologies (Fig.1). Statistics reveal that some of them have higher failure rates at younger age and have been rehabilitated earlier than others. This allows to identify types of pipe with different aging behavior and to estimate their service life under more ore less favourable conditions. These service life distributions can be expressed by mathematical functions and used in a cohort survival model (see next section) to forecast network rehabilitation needs. The cohort survival model can be used to generate a range of future rehabilitation needs under more or less optimistic assumptions (Herz 1998).

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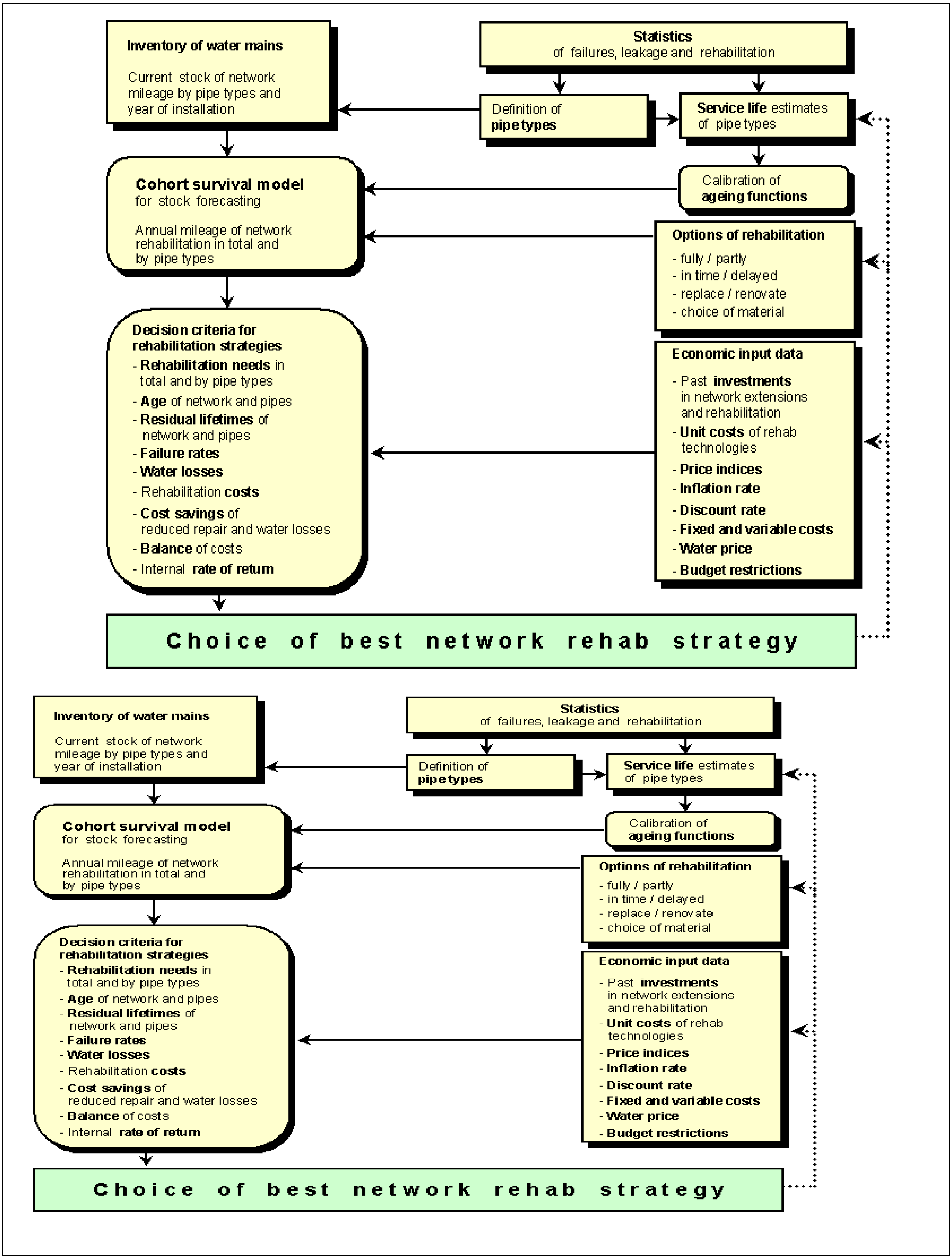



**Figure 1. KANEW framework for the exploration of network rehabilitation strategies**

The band of rehabilitation needs forecast by the cohort survival model for particular pipe types helps to define alternative rehabilitation programs. According to the rehabilitation program specified, the pipes that reach the end of their service life are rehabilitated, the others remain in the stock, get older and continue to deteriorate. The number of failures drop for the rehabilitated pipes whereas they increase for the others. So several decision criteria can be calculated year by year and expressed in economic terms by specific unit costs as shown in Fig.1. This allows to balance the costs and benefits of alternative rehabilitation programs. Because rehab investments come first and benefits later, accumulating over the service life of the new pipe, internal rates of return are calculated by discounting the costs and benefits of each rehabilitation program.

The best rehabilitation program will be found interactively by comparing systematically advantages and disadvantages of alternative programs and by generating new alternatives that come closer to the desired targets under budget restrictions.

## 3 A COHORT SURVIVAL MODEL FOR INFRASTRUCTURE DETERIORATION

The cohort survival approach stems from demography, where cohort survival analysis is widely used to forecast population changes from mortality and fertility statistics. Failures of infrastructure correspond to mortality and lead to rehabilitation. They tend to increase with age. Only some very old elements seem to last for ever. In an infrastructure survival model, types of elements installed or rehabilitated in the same year are called cohorts. As they move along the time axis, they get older and reach the end of their lifetimes according to their specific aging behavior. In cohort survival models, this progression in time is simulated successively for all cohorts year by year. Elements of infrastructure that have reached the end of their service life, usually are replaced, renovated or upgraded, constituting a rehabilitation need at a particular point in time.

This can be expressed in mathematical terms on the basis of Weibull, Gumbel and Herz distributions [Kleiner,Rajani 1999]. There are four types of aging functions that can be derived from these probability distributions:

- lifetime distribution function $f(x)$,
- survival function $1 - F(x)$,
- momentary failure/rehabilitation rate function $f(x) / (1-F(x))$ and
- residual lifetime expectancy function $R(x)$.

The <u>lifetime distribution</u> $f(x)$ is a probability density function defined for positive values of age x. It is usually bell shaped or skewed to the left. By definition, “infant mortality” of infrastructure elements is excluded as a matter of guarantee. For types of infrastructure that have been laid decades ago, lifetime distributions can be derived from statistics on the age of particular elements at the rehabilitation date. However, due to financial and technological changes, these statistics of past cohorts do not necessarily hold for the future. For new types of infrastructure, there is very little empirical evidence how long they will last.

<u>Survival functions</u> are derived from the cumulative $F(x)$ of the lifetime distribution. The percentage of elements of a cohort that reaches a particular age can be read from survival functions for specific types of infrastructure.

The <u>rehabilitation rate</u> of a cohort at a particular age is relating the elements that reach the end of their service life at a particular age to those that have survived so far. This rate is particularly useful in the simulation program because the stock of infrastructure is updated year by year. Again neglecting infant mortality of infrastructure elements, the rehabilitation rate starts from zero and gradually increases with age. A special feature of the Herz distribution (Herz 1996a,b) is that the rehabilitation rate is increasing progressively up to the median age, then turns into a degressive increase and finally approaches asymptotically a maximum rate. This means that the most resistant very old elements slow down their deterioration.

The <u>residual lifetime expectancy</u> starts from the mean value to decrease linearly at first, then degressively until it approaches a finite value close to zero, leaving some hope for survival. This results from the asymptotic behavior of the rehabilitation rate in the Herz distribution.

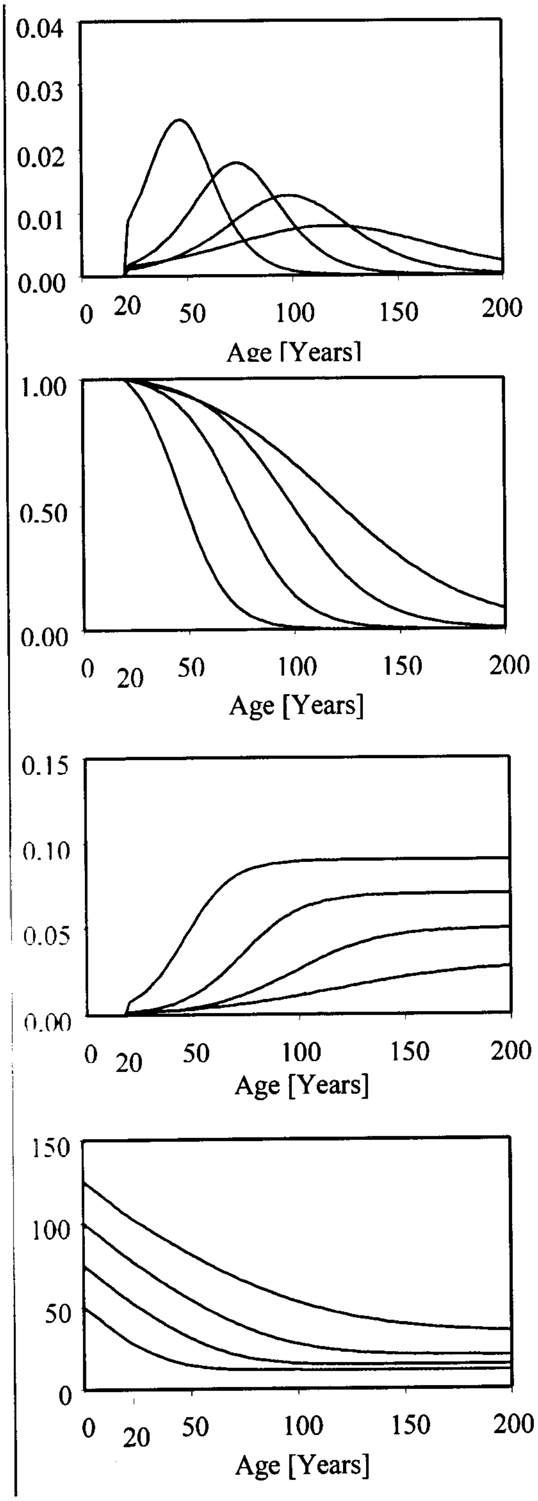
The Herz distribution is implemented in the KANEW model. It allows to calibrate specific aging functions for particular types of infrastructure by setting specific values for three aging parameters:

- aging factor $a$ ,
- failure factor $b$ and
- resistance time $c$ .

The <u>aging factor a</u> describes the smoothness of the starting phase of the aging process. The larger this value is, the smoother the aging process starts. For $a = 0$, the Herz distribution turns into the exponential distribution, which is not appropriate for describing the aging process of infrastructure elements because the failure rate starts abruptly and remains constant over the lifetime of the cohort.

<u>Failure factor b</u> is the final failure rate at very old age.

Up to <u>resistance time c</u>, there is no rehabilitation, just spot repair in case.




**Lifetime distribution**

$$f(t \le c) = 0$$

$$f(t > c) = \frac{(a+1)b\exp[b(t-c)]}{(a+\exp[b+(t-c)])^2}$$

$$f(t = \infty) = 0$$

$$E(t) = c + \frac{(a+1)\ln(a+1)}{ab}$$

**Survival function**

$$1 - F(t \le c) = 1$$

$$1 - F(t > c) = \frac{a+1}{a+\exp[b(t-c)]}$$

$$1 - F(t = \infty) = 0$$

**Failure/renewal rate**

$$z(t \le c) = 0$$

$$z(t > c) = \frac{b\exp[b(t-c)]}{a+\exp[b(t-c)]}$$

$$z(t = \infty) = b$$

**Residual lifetime expectancy**

$$E(\tau/t \le c) = c - t + \frac{(a+1)\ln(a+1)}{ab}$$

$$E(\tau/t > c) = X\frac{\ln X}{ab} - \left(\frac{t-c}{a}\right)$$

with $X = a + \exp[b(t-c)]$

$$E(\tau/t = \infty) = b^{-1}$$

**Figure 3. Aging functions of the Herz distribution with a resistance time of 20 years and lifetime expectancies of 50,75,100 and 125 years**

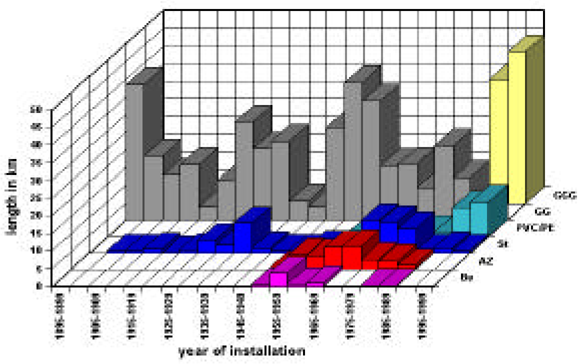
## 4 RESULTS FROM A GERMAN CASE STUDY

The water supply system of a city in East Germany is serving as an example to illustrate how future needs of network rehabilitation are forecast with KANEW and what the long term effects of a specific rehabilitation strategy will be. Although this case study is using data from a particular network and economic parameters which are typical for the German situation in general and this municipal water company in particular, some attempts are made to generalize this case study as much as possible.

### 4.1 Present situation

The water supply system is serving about 160, 000 people through 550 km of water mains plus 250 km of service pipes. Service pipes lie within the responsibility of the property owners. Water consumption of households plus industry is about 170 l/p·d. Before the German re-unification it was twice as much. About one third of the water fed into the system is non-revenue water, amounting to water losses in the order of 1 m³ per hour and km of water mains. Probably about two thirds of these water losses are due to bursts, leakage at cracks, corrosion holes, joints, valves and hydrants. The failure rate (with water extrusion) is about 0.55 per km and year for water mains and 2.35 for service pipes. So almost half of the leakage may occur on service pipes. These figures are significantly higher than the average for East German water distribution networks, which again is about twice as high as the West German average.

The future need of network rehabilitation depends on the aging behavior of the existing stock. In the following we concentrate on water mains (Fig. 3). Two thirds are cast iron (GG) pipes, most of them older than 50 years. After German re-unification, almost one hundred km of new ductile cast iron (GGG) pipes and some PE pipes have been laid. In addition to cast iron pipes, there are also some older steel (St), asbestos (Az) and concrete (Bs) pipes approaching the end of their service lives. During the last decade, almost 2% of the network has been rehabilitated per year, mostly by replacing old cast iron and steel pipes. There has been little cement-mortar relining of old cast iron pipes although most of them show heavy incrustation, but there were no problems with discolored water.

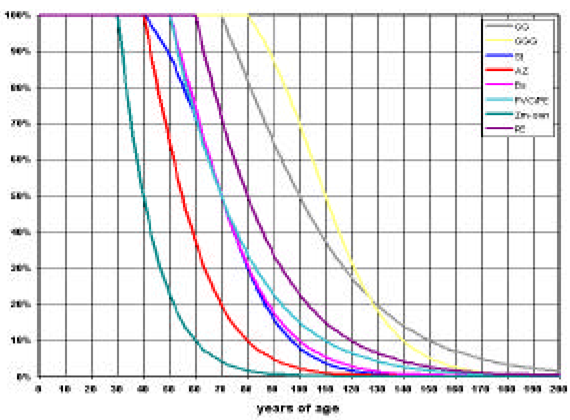



Seven categories of water mains are shown by 5-year periods of installation. Further distinctions could be made for pit cast iron and gray iron and for steel pipes of different quality and anti-corrosive protection. As there are always some pipes of unknown age and/or material, they may be assigned at best guess without causing major distortions to the calculation of future rehab needs.

**Figure 3. Length of water mains by period of construction and by material**

### 4.2 Aging behaviour of pipe types

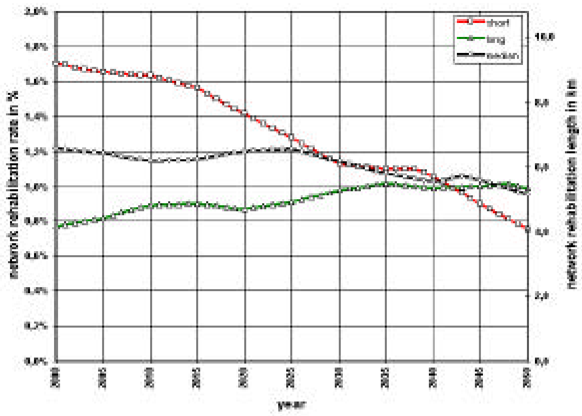
Pipe age alone cannot justify rehabilitation. However, age in combination with other characteristics of the pipe can be used to produce reasonable and consistent estimates of the service life of types of pipe (Fig. 4). Based on local experience and on the analysis of failure and rehabilitation statistics, ranges of age are estimated that would be reached by 100, 50 and 10 % of a particular type of pipe under more or less favourable circumstances. Usually this is done in an interactive way including research, engineering and management staff of the particular water supply company. For new types of pipe, the Delphi method is applied. Survival curves from other water utilities give some orientation.




As a rule, most water utilities do not replace a pipeline before it reaches a certain age. Up to this age, in case of a failure, there will be spot repair, no thorough rehabilitation. This age threshold varies with pipe material. Ductile iron pipes are assumed to be more resistant and to get older than cast iron and PE pipes. Renovation with cement mortar relining will prolong the service life of a pipeline under optimistic assumptions by 40 plus minus some years, for 10 % of them even by 60 years.

**Figure 4. Survival functions of types of water mains under optimistic service life assumptions**

The results of the cohort survival model depend very much on the estimates of specific service live distributions or survival functions. Therefore, simulation runs are performed by KANEW with lower and upper bounds and with median survival functions in order to delimit the range of future rehabilitation rates (Fig. 5).




Short service life assumptions lead to earlier and larger rehabilitation rates. In this case, they start with values that are about twice as high as those based on upper bound service life assumptions. With median service life assumptions, network rehab rates decline from 1.2 to 1.0 %. In most cases, pipe construction periods in the past result in pronounced peaks in the future. Quite often, particular types of pipes create specific peaks of rehab needs.

**Figure 5. Annual network rehabilitation rates assuming short, median and long pipe lives**

### 4.3 Defining rehabilitation programs

The forecast rehabilitation needs give some orientation for the rehabilitation program. In this case, the rehabilitation rates of 2% of the past decade would not be needed any more, even under pessimistic assumptions on the service life of the pipes. However, for the future rehabilitation program there are many options with different consequences. More or less pipes of a particular type can be rehabilitated more or less thoroughly earlier or later. This decision should be taken in the light of the effects that can be expected from specified rehabilitation measures, such as financial consequences and effects on burst and leakage rate, social cost, reliability of service, value of assets and, ultimately water tariff.

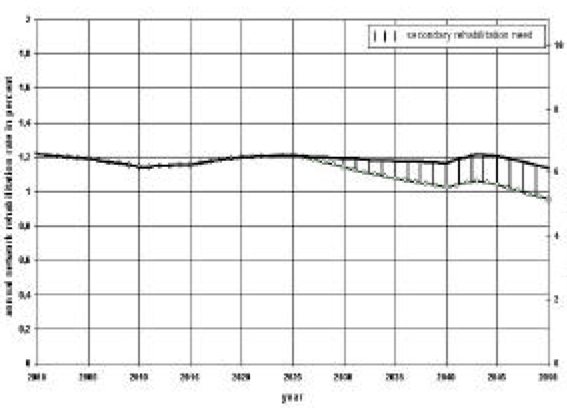
KANEW allows to specify annual rehabilitation rates for types of pipe within the rehabilitation period and, beyond this program horizon, calculates the over- or unfulfilled rehabilitation need of types of pipe up to time horizon of the analysis.

For this case study, a rehabilitation program was defined and tested that fulfils the network rehabilitation rate forecast on the basis of median service life distributions of types of pipes. In this rehabilitation program it is assumed that 20% of cast iron and steel pipes will be renovated, 60% will be replaced by ductile iron and 20% by PE-pipes. Old PVC/PE-pipes will be replaced by new more resistant PE pipes, concrete pipes by ductile iron pipes of larger diameters. 60% of asbestos pipes will be replaced by ductile iron, 40% by PE-pipes.

### 4.4 Forecasting the effects of a rehabilitation program

#### 4.4.1 Secondary rehabilitation needs

Any rehabilitation program will generate, on the long run, secondary rehabilitation needs from pipes which will rehabilitated in the future. This fact can be neglected for new pipes because they will need rehabilitation far beyond the time horizon of this analysis. However, pipes renovated by cement mortar relining will not last that long. The life of the old pipe is extended only by 30 to 40 years on the average. So within the forecasting period, renovated pipes will generate an additional secondary rehabilitation need which has to be taken into account. This secondary rehabilitation need cannot be fulfilled by another renovation. Thus, in line with the other specifications of this rehabilitation program it is assumed that 60% of these pipes will be replaced by ductile cast iron and 40% by PE-pipes. With median service life assumptions, this replacement process of renovated pipes starts after 25 years, and the network rehabilitation rate is increasing accordingly (Fig. 6).




Renovated pipes are assumed to survive at least 20 to 30 years (see Fig.4). Under median service life assumptions, half of them will reach an age of 35 years. So, in the year 2025 these renovated pipes start to create a secondary rehabilitation need, which is fulfilled by replacement. Thus low cost rehabilitation has a significant long term effect, in this case increasing network rehabilitation rate by 20% in 2050.

**Figure 6. Annual network rehabilitation rates under median pipe life assumptions including secondary rehabilitation needs of renovated water mains**

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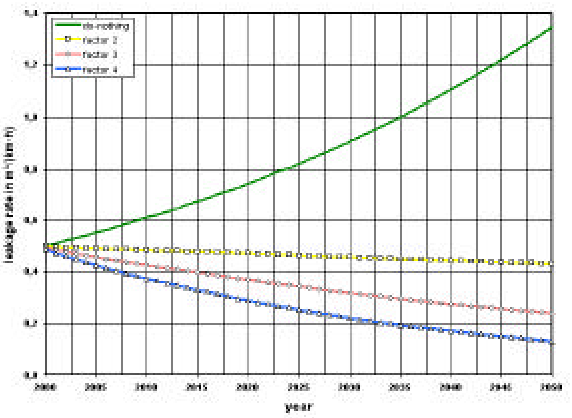
### 4.4.2 Leakage reduction

The leakage reduction resulting from this rehabilitation program is simulated by assuming that the new pipes will have minimal leakage rates of 0.01 m³ per km and hour which will increase by 2% per year. In the starting year 2000, the average leakage rate of water mains is 0.5 m³ per km and hour. The higher the rehabilitation rate, the more leakage is reduced.

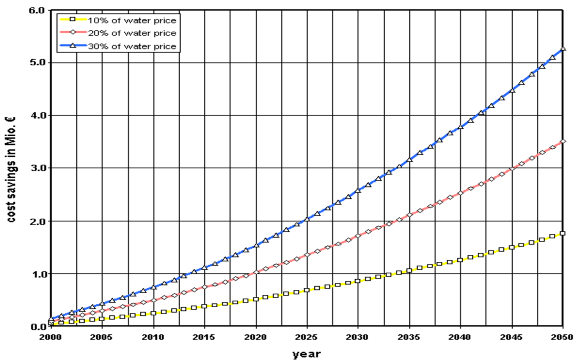



In the do-nothing alternative it is assumed here that leakage will increase annually at a constant rate of 2% on the average. Rates could be differentiated according to types of pipes. Rehabi-litation will reduce leakage to the extent of old leaky pipes being replaced by new and tight ones. Leakage reduction also depends on the efficiency of selecting the most leaky pipe-lines for rehabi-litation as expressed by a rehab efficiency factor related to the current network leakage rate.

**Figure 7. Leakage rates for do-nothing alternative and rehabilitation according to median service lives**

Leakage reduction also depends very much on how successful the network operators are in picking the most leaky pipelines for rehabilitation. When water mains are replaced in an integrated street reconstruction project including utility lines, leakage before replacement may be just about average. Rehabilitation programs that are especially designed for leakage reduction will select the most leaky pipelines. Empirical evidence from the water utility of the city of Chemnitz [Müller 1999] showed that water mains had leakage rates of about 3 times the network average before rehabilitation. This can be taken into account by a corresponding rehab efficiency factor. In order to show the effect of different rehab efficiency factors, the simulation model was run with rehab efficiency factors 2, 3 and 4 (Fig. 7). Note that this simulation shows only to leakage reduction from pipe rehabilitation. Spot repair of leaky pipes, possibly detected by special leakage reduction squads, will be more efficient in this respect, particularly for networks with high leakage.

The cost savings from leakage reduction depend on the quantity of saved water and its unit prize which is not the water rate(Herz,Hruza 1997). The results shown in Figure 8 are based on the defined rehabilitation program and a rehab efficiency factor of 3. In general, water prevented from leaking out is worth less than if it is sold to customers because most of the production cost are fixed interest and personnel costs. Cost savings are marginal costs accruing from the variable unit costs of water production. Only exceptionally, when leakage reduction allows to close an existing production unit or to avoid the construction of new one, marginal cost savings will be higher than variable costs. Without information on marginal cost in this particular local situation, the share of variable costs is taken for marginal pricing of leakage reduction. The local water price, which covers the fixed and variable costs of water supply, is 2 €/m³ in the year 2000 and is estimated to rise by 1% per year.




As leaky pipes are replaced year by year, and have only minimal leakage in subsequent years, the volumes of saved water accumulate year by year. They are valued by the variable part of the water price (10, 20, 30%) as a proxy for the marginal costs of saving water. This part of the water price covers only those cost elements which are directly related to the quantity of sold water.

**Figure 8. Cost savings from reduced water leakage for different marginal costs**

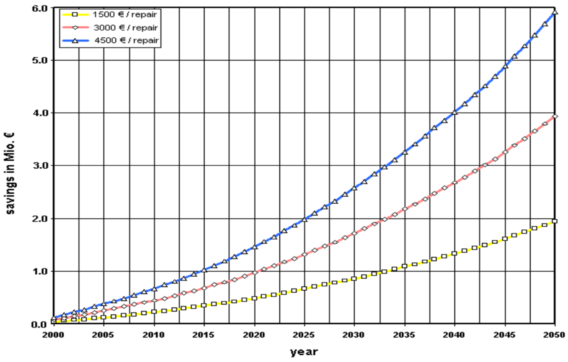
### 4.4.3 Reduction of failures and repair

In addition to cost savings from reduced leakage, benefits accrue from reduced repair work on new pipelines in particular and on renovated ones as well. The effect on failure reduction is simulated in the same way as for leakage reduction. It is assumed that pipelines with relatively high failure rates are chosen for rehabilitation and that new pipelines will have a failure rate of 0.01 and renovated ones 0.1 failures per km and year increasing by 2 % per year.

In this case study, the starting failure rate for water mains is 0.55 failures per km in the year 2000. Failures are assumed to increase annually by 2 % if no rehabilitation measures were taken and failure prone pipelines to be chosen with an efficiency factor of 3.

Average repair cost is 1500 € per failure at present and assumed to rise with 1.5% per year. This figure should cover all direct costs of the water utility, including the fees paid to the insurance company for compensation of damage caused by pipe failures. However, it certainly does not include external costs of service interruption, traffic congestion due to burst pipes and repair work in the street, noise and dust emission on passengers and neighbors nor compensation for temporary sales reduction in adjacent shops and services. These indirect, external or social costs are usually taken into account by applying factors to the direct costs.

In this example, social cost factors of 2 and 3 are applied in order to show the effect (Fig. 9).

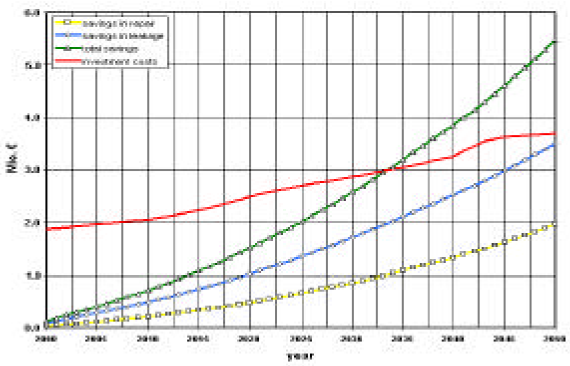



As new pipes have minimal failure rates in subsequent years, repair cost savings accumulate in a progressive way. Whereas the water utilities cover only the direct costs, the public may be concerned with the indirect "social" costs which are associated with repair work in the streets. Social cost factors of 2 and 3 times the direct costs show corresponding effects on the benefits from savings in repair costs.

**Figure 9. Cost savings from reduced failures for different social cost factors**

4.4.4 Rehab investments

Annual rehab investments are calculated by multiplying the annual pipe replacement and renovation with their specific unit costs. In this case, the unit costs for replacement are 300 € per meter of ductile iron pipes and 275 € for PE-pipes. The unit cost for cement mortar relining is 225 €/m. Prices for pipe work are assumed to rise by 1.5% per year. With these economic data, the annual investment costs and benefits of this particular network rehab program can be presented up to the year 2050. The benefits shown in Figure 10 are calculated with an efficiency factor of 3. Reduction of leakage is valued with a marginal cost of 20 % of the water price, and reduction of repair is only for direct costs.



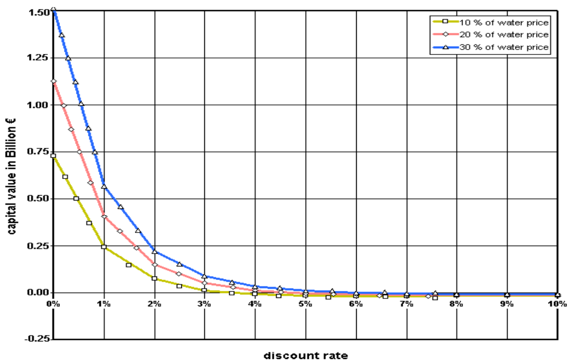

Annual rehab investments increase rather steadily up to the year 2050. The benefits from savings in leakage and repair costs accumulate and surpass rehab investments in the year 2036. Beyond the year 2050 no further rehab investments are considered, whereas benefits from past rehab investments will continue to accrue, although in a decreasing manner.

**Figure 10. Annual rehab investments and cost savings from reduced water leakage**

4.4.5 Comparing costs and benefits

As can be seen from Fig. 9, for this particular rehabilitation program, the benefits from cost savings in leakage and repair are reaching the same order of magnitude as the rehab investments. There is a time lag and a cumulative effect of benefits. In this case, savings from reduced leakage contribute more than those from reduced repair. This depends on the marginal cost of water leakage and the unit cost of repair. Up to the year 2035, rehab investments are higher than the cost savings. The year 2036 could be called "break even year". Thereafter, benefits from savings are larger than the costs of rehab investments. It should be noted that the benefits from rehab investments extend beyond the year 2050 by the service life expectancy of rehabilitated pipelines, particularly ductile cast iron pipes, which this are assumed to last 100 years on the average.

Benefits which occur in the far future do not have the same weight as costs which accrue in the next years. Therefore, costs and benefits are discounted into present values. As the present values of costs and benefits depend heavily on the discount rate (see Fig. 11), internal rates of return are calculated for which the discounted benefits are equal to the discounted costs. Due to the fact that rehab benefits extend beyond the period of investment, there are significant net benefits at low discount rates. Rehab investments pay off as long as future net benefits are discounted with rates beyond the internal rates of return. In general, it is recommended that the internal rate of return should be at least 3% larger than the inflation rate. Thus the rehab investments of the given example would pay off.




The higher the marginal costs of water losses, the larger are the present values of the discounted net benefits and the internal rate of return. Investments pay off if the internal rate of return is larger than the inflation rate. In this case, annual inflation was assumed to be 1.5% for pipe works, which is significantly lower than the internal rates of return for savings in direct repair costs and leakage valued at 10 to30% of the water price.

**Figure 11. Internal rate of return for rehab investments with median service lives**

## 5 CONCLUSIONS AND OUTLOOK

The aim of this paper was to show how the long-range effects of specified rehabilitation programs can be analyzed in a comprehensive cost benefit framework and used to develop an "optimal" network rehabilitation program. The process of network deterioration and the effects rehabilitation needs that are not fulfilled in due time, were calculated with a cohort survival model. With this model, as implemented in the KANEW software, several network performance indicators were forecast year by year and expressed in economic terms. Assumptions have to be made for such long term forecasts and economic evaluation. The sensitivity of the results with respect to the uncertainty of specific assumptions is shown for an illustrative example of an East German water supply system.

Some general conclusions may be drawn from this study. In the first place, such long-term projections appear to be a useful complement to the "fire brigade" and monitoring approaches prevailing in water supply network management. In spite of our limited forecasting capabilities, the band of rehabilitation needs can be determined by a cohort survival model for specific types of pipes in the near and far future. Cohort survival models are implemented in KANEW, WARP and the Nessie Model. Such models should be used for water company asset management.

Second, rehabilitation programs for a period of 5 to 10 years should be developed after looking at the effects they are expected to have in the short, medium and long run. In the short run they will not pay off, because the benefits extend and accumulate over the whole service life of the rehabilitated pipes. However, the intensity of rehabilitation work that can be justified in economic terms varies considerably with the present state of the network, the water rate and the direct and indirect costs of repair and rehabilitation. With low water prices and cheap repair of broken pipes and leaks, rehab investments will not pay off, even in the long run, if external costs of pipeline failure and repair are not included. These so-called social costs, which are difficult to quantify in general, enlarge the benefits from rehabilitation (Herz 1995, Baur,Herz 1999).

Third, due to the large number of network rehabilitation options, an "optimal" rehabilitation program has to be developed in an interactive way by modifying those parameters that lead to undesired results. This implies a formal procedure by which the outcome of alternative rehabilitation programs is systematically compared with each other. Such a multi-criteria evaluation procedure is being developed in the CARE-W project.

The approach presented in this paper is a top down approach, which is recommended for strategic investment planning and asset management. The focus is on the network, not on individual pipes within the network. The efficiency of a network rehabilitation program depends very much on the efficiency of the individual pipeline rehabilitation projects included in the program. So, in a top down approach, the next step of selecting the most cost-efficient rehabilitation projects is a very important one. Research is under way on this subject in CARE-W as well (Baur/Kropp 2001).

## 6 ACKNOWLEDGMENTS

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# The Effect Of Temporal And Spatial Variability Of Ambient Carbon Dioxide Concentrations On Carbonation Of RC Structures

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**Summary**: **Ambient carbon dioxide ($CO_2$) is an environmental stressor that can lead to the carbonation and deterioration of RC structures. In the present paper a useful model to predict carbonation depths is reviewed. The model uses $CO_2$ concentration as an input parameter, but there appears to be a lack of quantitative data on existing $CO_2$ concentrations in typical urban or industrial cities. Hence, the paper reports on $CO_2$ concentrations collected from the city of Brno in the Czech Republic. It was found that the ambient $CO_2$ concentration attributable to a typical urban environment is approximately 5-10 % higher than $CO_2$ concentrations in a rural environment. Carbonation depths were calculated for RC structures with service lives of up to 100 years. It was seen that the increase in urban and global $CO_2$ concentrations have little influence on these predicted carbonation depths. Using a minimum (worldwide rural) $CO_2$ concentration of 370 ppmv for exterior exposures and assuming that material and environmental properties are known, will provide a lower bound on the actual depth of carbonation. Finally, a probabilistic analysis showed that variability in carbonation depths can he high due to uncertainty and variability of environmental and material properties.**

**Keywords. Carbonation, Probabilistic Analysis, Concrete, Deterioration, Variability**

## 1 INTRODUCTION

The durability of reinforced concrete (RC) structures is, not surprisingly, adversely affected by environmental stressors. A rather common and serious stressor is carbon dioxide ($CO_2$) which can cause depassivation of the protective film of steel reinforcement (known as carbonation). Carbon dioxide is always present in the atmosphere and its concentration is higher in the vicinity of its sources – in industrial and densely populated regions which tend to have the highest proportion of built infrastructure. Being heavier than air higher concentrations of $CO_2$ may be expected to vary with height above ground level.

When the carbonation depth reaches the reinforcement the reduction in alkalinity destroys the passivity of the protective film - corrosion may then occur. The time interval to this situation is normally referred to as the initiation period. During the following time (propagation) period the corroding steel sections are gradually reduced, leading to cracking and finally spalling of concrete thus reducing the structural and serviceability reliability of structural members. An appropriate definition of service life would most likely be the time to significant corrosion-induced cracking and spalling [e.g., Stewart and Rosowsky, 1998; Stewart, 1999, 2001]. However, the present study will focus on a more conservative measure of service life performance, namely, the time to corrosion initiation.

The measurement of $CO_2$ concentrations are seldom reported or readily accessible, are rarely based on continuous monitoring and are more likely to be recorded in non-urban (remote) environments so as to monitor the background level of $CO_2$ concentrations needed to help assess the magnitude of "global warming". Therefore the aim of the present paper is (i) to report about actual $CO_2$ measurements recorded in typical urban environments and (ii) to show the damaging effect of $CO_2$ on RC structures and the variability of predicted carbonation depths.

## 2 PREDICTIVE MODEL FOR CARBONATION OF CONCRETE

The time to corrosion initiation (carbonation) depends on many parameters: concrete quality, concrete cover, relative humidity, ambient carbon dioxide concentration and others. The impact of carbonation has been studied by several researches and various mathematical models have been developed with the purpose of predicting carbonation depths. Most models predict that carbonation depths increase as a function of the square root of time. Some of the carbonation models are listed and compared in Kersner, et.al. (1996) and others are described in Papadakis, et.al. (1992), Bickley (1990), Ho and Lewis (1987), Richardson (1990), Schliessl (1976), etc. However, the model proposed by Papadakis, et.al. (1992) considers the widest range of influencing parameters and is the only model where the $CO_2$ concentration is directly involved as an input parameter. The mathematical model is based on differential mass-balances of gaseous $CO_2$, solid and dissolved $Ca(OH)_2$, CSH and unhydrated silicates, which account for the production, diffusion and consumption of these substances. The carbonation depth ($x_c$ in m) is thus

$$x_c = \left( \frac{2[CO_2]^0 D_{e,CO_2}}{[CH] + 3[CSH]} t \right)^{1/2} \qquad RH \geq 50\% \tag{1}$$

$$[CO_2]^0 = 42 y_{CO_2} 10^{-6} \tag{2}$$

$$D_{e,CO_2}(m^2 yr^{-1}) = 51.8 \varepsilon_p^{1.8} [1 - (RH/100)]^{2.2} \tag{3}$$

$$\varepsilon_p \approx \left( \frac{\rho_c}{\rho_w} \right) \frac{(w/c) - 0.3}{1 + (\rho_c/\rho_w)(w/c)} \tag{4}$$

$$[CH] + 3[CSH] \approx \frac{33000}{1 + (\rho_c/\rho_w)(w/c) + (\rho_c/\rho_a)(a/c)} \tag{5}$$

where $[CO_2]^0$ is the molar concentration of ambient $CO_2$ (mol $m^{-3}$); $D_{e,CO2}$ is the effective diffusity of $CO_2$ in concrete; [CH]+3[CSH] is the total molar concentration of carbonable constituents of concrete (mol $m^{-3}$); t is time in years; $y_{CO2}$ is the ambient $CO_2$ content by volume (ppmv); RH is the relative humidity (%); $\varepsilon_p$ is the porosity of fully hydrated and carbonated cement paste; $\rho_c$, $\rho_w$ and $\rho_a$ are the densities of cement, water and aggregates respectively ($kg/m^3$); and c, w and a are the contents of cement, water and aggregates respectively (kg). The aggregate content is the sum of sand and gravel (sizes 4-8mm, 8-16mm) contents. Clearly, relative humidity and ambient $CO_2$ content are long-term averages (ie. time-invariant) values, say measured over a year or more.

Papadakis, et.al. (1992) then proposed a "simplified" expression for carbonation depth, namely,

$$x_c \approx 350 \left( \frac{\rho_c}{\rho_w} \right) \frac{(w/c) - 0.3}{1 + (\rho_c/\rho_w)(w/c)} f(RH) \sqrt{\left[ 1 + \left( \frac{\rho_c}{\rho_w} \right)(w/c) + \left( \frac{\rho_c}{\rho_a} \right)\left( \frac{a}{c} \right) \right] \left( y_{CO_2} 10^{-6} \right) \times t} \tag{6}$$

where f(RH) is a function of relative humidity equal to 1-(RH/100). However, Novák, et.al. (1996) have proposed a step-wise linear relationship for f(RH) extracted from field data of a cooling tower in Bohemia. Note that Eqn. (6) is somewhat non-conservative since it underestimates carbonation depths by approximately 5-10%. For this reason, Eqns. (1)-(5) are the preferred expressions for the calculation of carbonation depths to be used in this paper, for indoor conditions or outdoor concrete sheltered from rain.

# 3 TEMPORAL AND SPATIAL VARIABILITY OF $CO_2$ CONCENTRATIONS

## 3.1 Temporal Variability and Global Warming

It has been recognised for some time that $CO_2$ concentrations are subject to temporal and spatial variability. The average $CO_2$ concentrations recorded in the South Pole and other unpolluted atmospheres have steadily increased from 330ppmv in 1971 to approximately 370 ppmv in 2000 [e.g., Keeling and Whorf, 2000]. This represents an annual increase of over 1.4 ppmv per year. As a comparison, the pre-Industrial Revolution level of $CO_2$ concentration was between 265 and 290 ppmv [Engelfried, 1985]. However, this increase in average $CO_2$ concentrations might just be an increase in the "global average" $CO_2$ concentration and may not be representative of increases in previously established urban environments. In other words, the increase in "global average" $CO_2$ concentrations may be due more to development of new and expanding urban and industrial regions (e.g., in the developing world), rather than an increase in $CO_2$ concentrations within existing urban boundaries (e.g., within a city whose level of inter-decadal $CO_2$ emissions would be relatively time-invariant). There appears to be little data to confirm this hypothesis. However, Fig. 1 shows that annual average $CO_2$ concentrations in Calgary (Canada) measured from 1992 to 1999 vary from 390 ppmv to 400 ppmv (mean is 394 ppmv with an annual increase of 1.4 ppmv) [CASA, 2001]. It is unclear whether the increase in annual average $CO_2$ concentrations during this period is statistically significant or not. The considerable temporal variability within a year is shown in Fig. 2. Longer term records are needed, but unfortunately, collection of $CO_2$ concentration data in Calgary seems to have ceased in 2000.

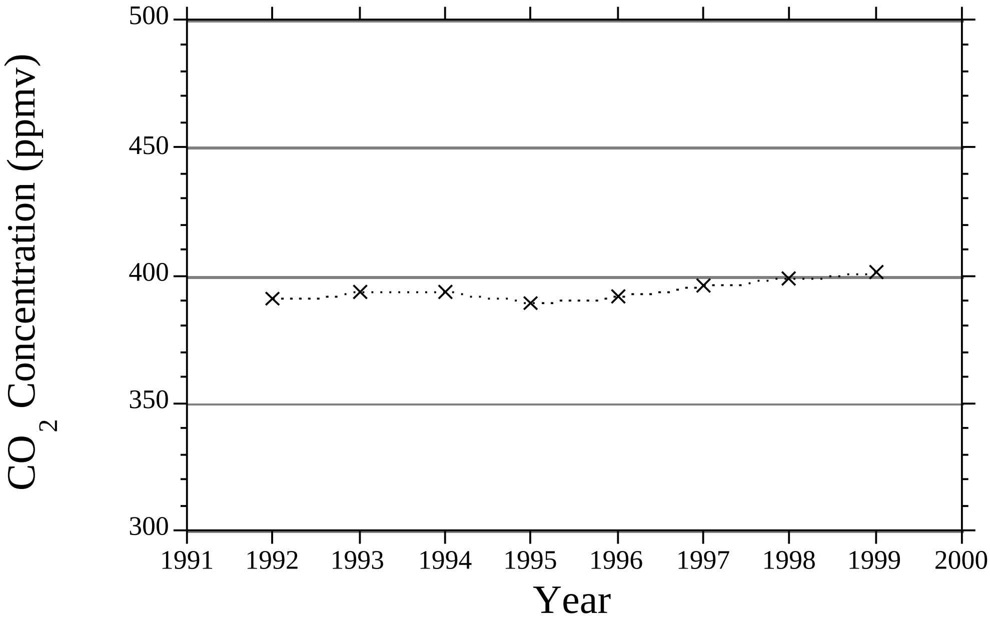


**Figure 1. Average Annual $CO_2$ Concentrations for Calgary, Canada [CASA, 2001].**

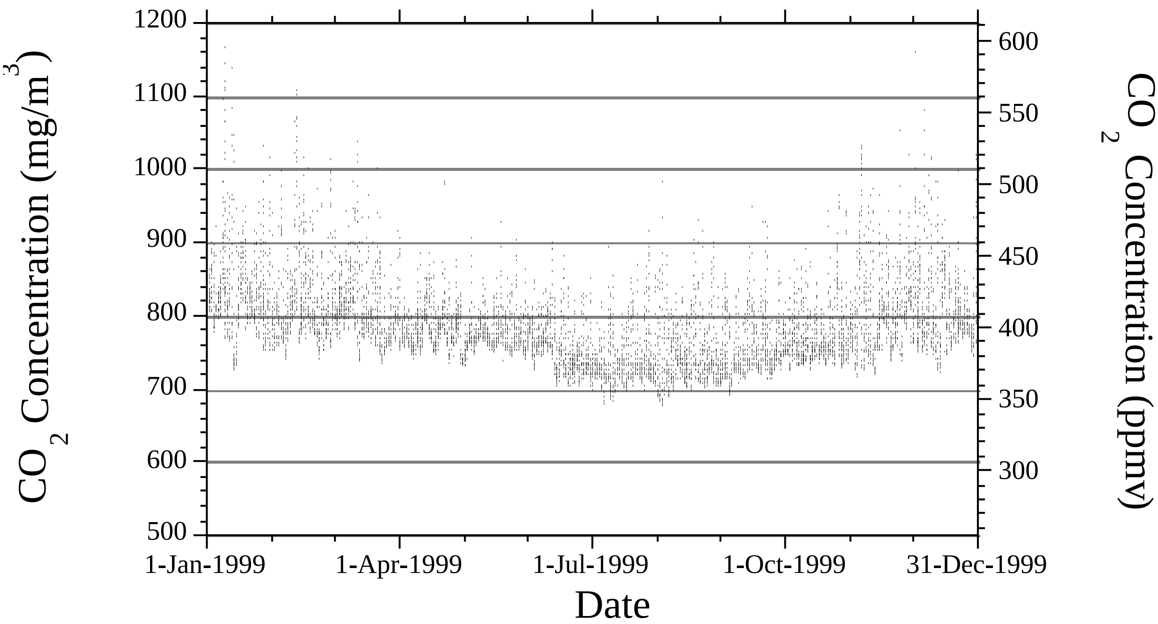


**Figure 2. Temporal Variability of Hourly $CO_2$ Concentrations for Calgary [CASA, 2001].**

### 3.2 Measurements from Meteorological Station, Brno University of Technology

Long-term $CO_2$ concentration measurements in urban environments are relatively scarce. For this reason, the Meteorological Station of the Faculty of Civil Engineering, Brno University of Technology started to monitor hourly $CO_2$ concentrations in April 1999. Carbon dioxide is measured by a Multi Gas Monitor Type 1302 (Bruel&Kjaer) using the photo-acoustic infra-red detection method.

Carbon dioxide concentrations were initially measured from the sixth floor (30m above a relatively busy street 2km from the city centre) of the Meteorological Station. Because of the reconstruction of the building containing the Meteorological Station in September 2000 the $CO_2$ monitor was moved to an external location on the ground floor (2m above street level). Monitoring is still continuing at this location, although the Meteorological Station will eventually be relocated to its original position. For example, Fig. 3 shows hourly monitored $CO_2$ concentrations for a typical day for (i) 30m above street level and (ii) 2 m above street level. It is observed that the $CO_2$ concentration varies quite significantly over a 24 hour period. The Meteorological

Station is not located near any industries so it is expected that hourly changes in $CO_2$ levels are attributable mainly to exhaust emissions, especially with morning temperature inversion and during peak hours.

From all the measured data collected to date the minimum hourly $CO_2$ concentration was 350 ppmv and the maximum recorded value being 575 ppmv. These results are consistent with individual hourly measurements made in Calgary (see Fig. 2). The daily average $CO_2$ concentration (in year 2000) is 390 ppmv for the sixth floor and 408 ppmv for 2m above street level. In Phoenix (Arizona) $CO_2$ concentrations measured at a height of 2m varied from 555 ppmv in the city centre to 370 ppmv in outlying rural areas [Idso, et.al., 1998]. This suggests the presence of "urban $CO_2$ domes" where intensive urbanisation can increase local $CO_2$ concentrations. It might be noted that the average $CO_2$ concentrations recorded in the South Pole, Pacific Islands, Alaska and other unpolluted atmospheres are approximately 370 ppmv (725 mg/m$^3$). This suggests that the minimum average $CO_2$ concentration is 370 ppmv for exterior exposures.

The concentration of $CO_2$ in urban environments are influenced by combustion of fossil fuels from traffic, domestic heating, power generation, etc. Carbon dioxide concentrations at higher levels could also be influenced by wind velocity and wind direction, together with air turbulence. During calm weather $CO_2$ produced by traffic is concentrated close to ground level and $CO_2$ has a density 50% greater than air which helps explain the higher $CO_2$ concentrations in Brno recorded at 2m above street level. Concentrations of $CO_2$ are increased also by air temperature inversion. This occurs especially in low lying urban localities during morning hours (see Fig. 3). Note that the $CO_2$ concentration for indoor environments can be considerably higher than exterior $CO_2$ concentrations.

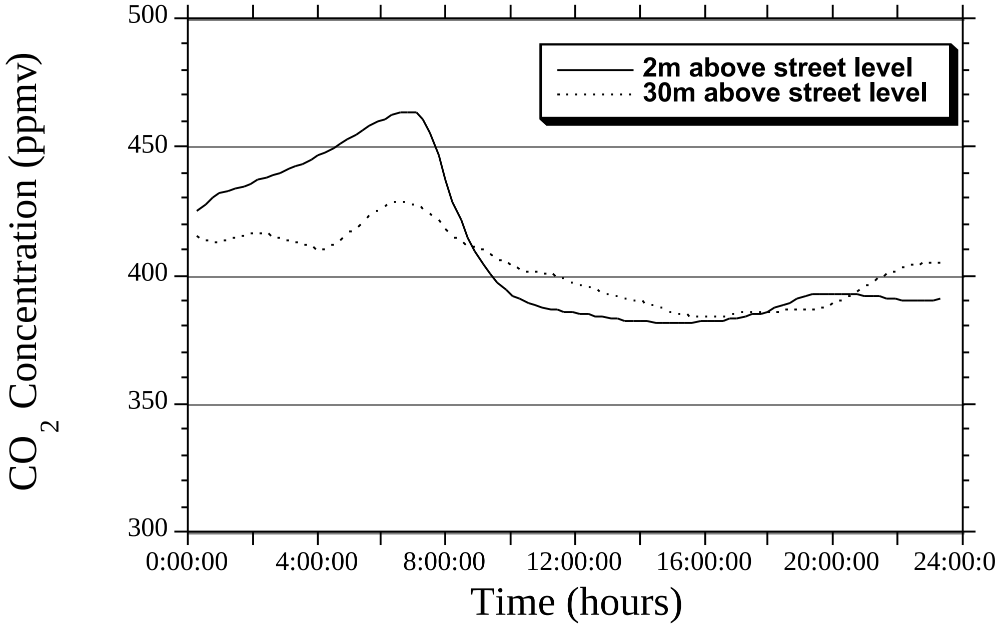


**Figure 3. Hourly $CO_2$ Concentrations Recorded for a Typical Day in Brno.**

More extensive predictive and statistical analyses will be feasible when more data is collected from the Meteorological Station of the Faculty of Civil Engineering, Brno University of Technology. This will allow, for example, analysis of seasonal $CO_2$ changes as well as investigating the relationship between $CO_2$ concentrations and other meteorological (environmental) factors. It is hoped that additional $CO_2$ monitoring stations can be commissioned so as to allow for greater understanding of the temporal and spatial variability of $CO_2$ concentrations.

## 4 PREDICTED CARBONATION DEPTHS

### 4.1 Deterministic Modelling

The influence of $CO_2$ concentration on the carbonation front ($x_c$) as calculated from Eqns. (1)-(5) is shown in Fig. 4; for typical concrete of class C30/37, RH equal to 65%, and $CO_2$ concentrations of 370 ppmv, 400 ppmv and 450 ppmv. The impact of $CO_2$ concentration is clearly observable – e.g. a carbonation depth of a low cover (10mm) could be reached in only 32 years for a known $CO_2$ concentration of 400 ppmv (0.04% $CO_2$ by volume). It was shown earlier that the minimum $CO_2$ concentration is 370 ppmv for exterior exposures and so this may be deemed a "best case" scenario for durability. In other words, using a $CO_2$ concentration of 370 ppmv (725 mg/m$^3$) in Eqn. (1), and assuming that material and environmental properties are known, will provide a lower bound on the actual depth of carbonation. The present analysis assumes that $CO_2$ concentration is modelled as a stationary (time-invariant) process. However, if $CO_2$ concentrations are increasing with time then $CO_2$ concentration needs to be modelled as a non-stationary process [e.g., Fukushima, et.al., 1996].

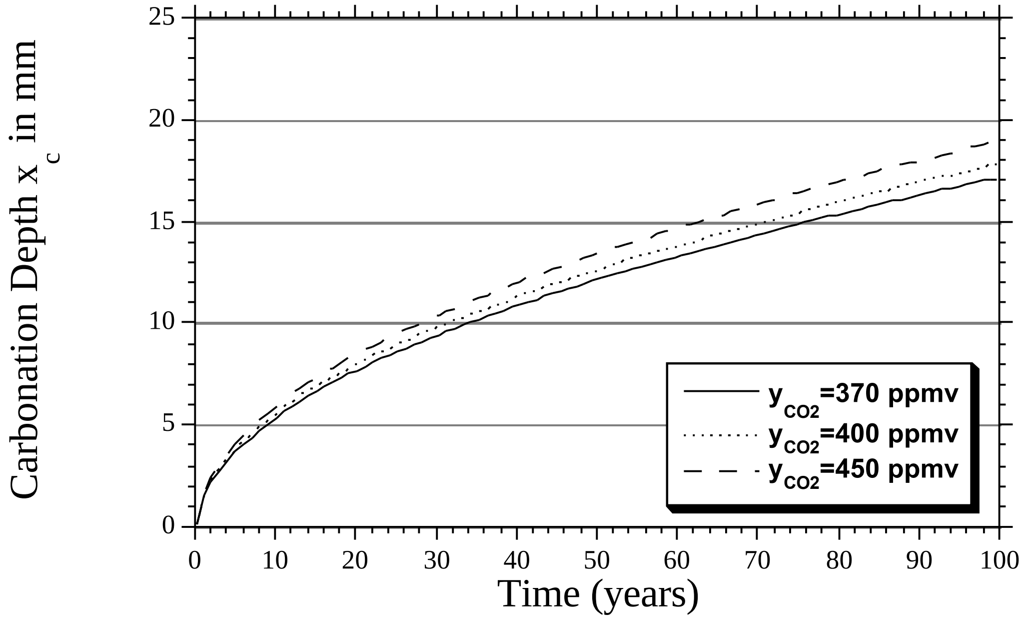


**Figure 4. Function of Carbonation Depth with Time**

If it is assumed that annual $CO_2$ concentrations increase at 1.4 ppmv then the average $CO_2$ concentration over 100 years will be 470 ppmv. In this case, the carbonation depths for this $CO_2$ concentration are about 8% higher than the present urban $CO_2$ concentration (400 ppmv). This represents an approximation of carbonation depths since a more accurate analysis would model $CO_2$ concentration as a non-stationary process increasing linearly from 400 ppmv to 540 ppmv over the next 100 years.

The collected data shows that the additional ambient $CO_2$ concentration attributable to a typical urban environments is approximately only 20-40 ppmv. For a "best case" scenario of 370 ppmv this represents an increase in $CO_2$ concentration of 5-10%. This change in $CO_2$ concentration will cause less than a 5% increase in carbonation depths. This suggests, that for carbonation, changes in relative humidity and concrete quality have a greater influence on durability of RC structures than the additional $CO_2$ concentration experienced in typical urban environments or that expected from global warming. This later observation is supported by Engelfried (1985) who stated that the additional carbonation caused by "polluted" atmospheres is minor.

### 4.2 Probabilistic Modelling

It is widely recognised that many parameters involved in determining carbonation depth are uncertain or random in nature. So it is useful to adopt a probabilistic analysis by treating input parameters as random variables or processes [e.g., Stewart and Melchers, 1997]. This approach, for carbonation depths, was initiated probably by Hergenroder (1988) and later utilised by Teply, et.al. (1993), Holicky and Mihashi (1999), and others. A probabilistic analysis propagates the uncertainty and variability of these parameters to provide an indication of the variability of the final result, in this case, the mean and variance of carbonation depths. Monte Carlo simulation is useful for such a probabilistic analysis.

Statistical parameters (mean, coefficient of variation, distribution type) for class C30/37 concrete exposed to an exterior environment are described in Table 1. The model error is modelled as an annual random process with a correlation coefficient of 0.5. Considering model error as a random process will help account for the temporal variability of the accuracy of predictive models. Relative humidity and $CO_2$ concentrations might also be considered to be non-stationary and highly correlated annual random processes; however, these are not considered herein.

**Table 1. Statistical Paramters of Carbonation Variables.**

| *Variable* | *Symbol* | *Mean* | *Coefficient of Variation (COV)* | *Distribution* |
|---|---|---|---|---|
| *Random Processes:* | | | | |
| Model Error | $ME(t_o)$ | 1.0 | 0.1 | Normal |
| *Random Variables:* | | | | |
| Relative Humidity | RH | 65% | 0.05 | Normal |
| Cement Content | c | 370kg | 0.03 | Lognormal |
| Water Content | w | 181kg | 0.02 | Lognormal |
| Sand Content | s | 644kg | 0.03 | Lognormal |
| Gravel (4-8) Content | $g_{4-8}$ | 262kg | 0.03 | Lognormal |
| Gravel (8-16) Content | $g_{8-16}$ | 925kg | 0.03 | Lognormal |
| Cement Density | $\rho_c$ | $3100kg/m^3$ | 0.03 | Normal |
| Sand Density | $\rho_s$ | $2590kg/m^3$ | 0.03 | Normal |
| Gravel (4-8) Density | $\rho_{g4-8}$ | $2540kg/m^3$ | 0.03 | Normal |
| Gravel (8-16) Density | $\rho_{g8-16}$ | $2660kg/m^3$ | 0.03 | Normal |

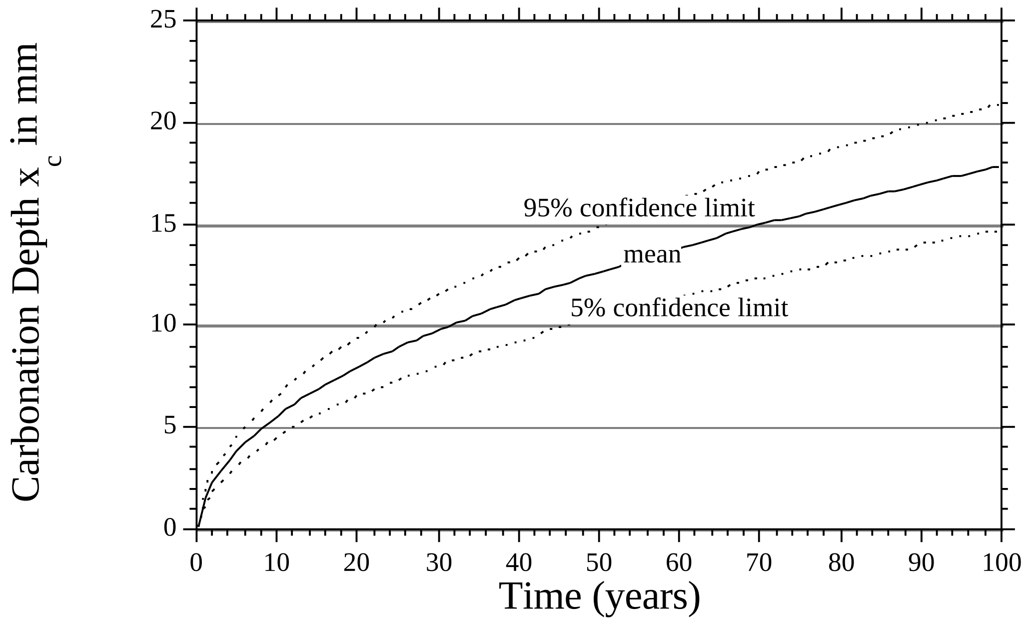


**Figure 5. Variability of Carbonation Depth ($y_{CO2}$=400ppmv)**

Fig. 5 shows the variability of carbonation depths for a $CO_2$ concentration of 400 ppmv. The scatter in carbonation depth is quite large (COV=0.14), as is also usually observed in reality. In the present case, $CO_2$ concentration is treated as a deterministic variable. However, if the variability of carbonation depth is needed for a population of structures exposed to different $CO_2$ concentrations (e.g., different building heights) then a moderate variability of $CO_2$ concentration (COV less than 0.15) will produce little influence on carbonation depths. If such an analysis was conducted then the probabilistic model of $CO_2$ concentrations would need to be truncated at 370ppmv. Not surprisingly, a sensitivity analysis has found that the most significant variables are relative humidity and model error.

### 4.3 Chloride-Induced Corrosion

It should be noted that chloride ingress also jeopardises the passivation layer on reinforcing bars. The modelling of this effect can be done using a number of predictive models [e.g., Papadakis, et.al., 1996; Vu and Stewart, 2000] and have been utilised for probabilistic assessment of time to corrosion initiation of concrete structures [Teply, et.al., 1998; Weyers, 1998; Vu and Stewart, 2000]. There is evidence to suggest that chloride action is accelerated by carbonation (and $SO_2$, $NO_x$) because carbonation disturbs the equilibrium between free and bound chlorides in the concrete, thereby increasing the free chloride concentration in the pore solution. The interaction between carbonation and chloride-induced corrosion may be significant; unfortunately, a suitable predictive model is not available.

## 5 CONCLUSIONS

Continuous monitoring of carbon dioxide concentrations in Brno has shown that the ambient $CO_2$ concentration attributable to a typical urban environment is approximately 5-10% higher than $CO_2$ concentrations in a rural environment. Measurements from Calgary in Canada confirm this observation. Carbonation depths were calculated for RC structures with service lives of up to 100 years and it was seen that increased urban $CO_2$ concentrations have little influence on these predicted carbonation depths. Using a minimum (worldwide) $CO_2$ concentration of 370ppmv for exterior exposures will represent a “best case” scenario or lower bound for durability. A probabilistic analysis showed that variability in carbonation depths can he high due to uncertainty and variability of environmental and material properties. Monitoring is continuing and should be extended to include other localities to gain a better understanding of the spatial characteristics of $CO_2$ concentrations, and hence, the extent of carbonation in new and existing RC structures.

## 6 ACKNOWLEDGMENTS

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# Estimating The Service Life Of Jointing Products And Systems
# Application Of A Crack Growth Model To Different Climates

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**Summary**: **Models for service life estimation of jointing systems and products are potentially useful for comparisons among the relative performance of different products in specific systems, similar comparison of products in different climates, or for helping establish requirements for maintenance and refurbishment of building envelopes. Although generic methods for developing service life estimates exist in literature, no practical and verifiable models have yet been developed for jointing products. However, notional models for products have been proposed based on damage functions related to fatigue rupture and crack growth and provide a basis for further work. This paper reports on the use of a crack growth model to assess crack development over time when subjected to different types of climates. The effects of installation temperature and relative humidity on potential crack growth in joints located in various cities are considered. Estimates of expected service life are provided in relation to specific performance criteria.**

**Keywords. Climate, crack growth model, damage function, jointing systems, sealant, service life model**

## 1 INTRODUCTION

Wolf [2000a, b] states that service life models for jointing products and systems have yet to be developed despite the fact that deterioration models for sealant products do exist. Indeed there is an evident lack of information that relates deterioration induced from laboratory testing to that observed in field inspections. Systematic approaches to service life prediction for building materials and components are available from both RILEM [Masters & Brandt 1989] and ASTM [1996] technical documents. As well, these proposed methods are both consistent with the requirements outlined by Wolf [2000a] and are readily applicable to addressing the apparent lack of mathematical connection between laboratory and field exposure results.

Both documents recommend that service life prediction be based on a comparison between the performance over time of degradation functions derived from either short-term or accelerated tests and that of long-term exposure tests. This generic approach (Figure 1), adapted to jointing systems [Lacasse & Cornick 2001] suggests that a service life or performance over time model incorporates information derived from both deterioration or damage models, as well as life models derived from in-situ inspections on real joints in buildings.

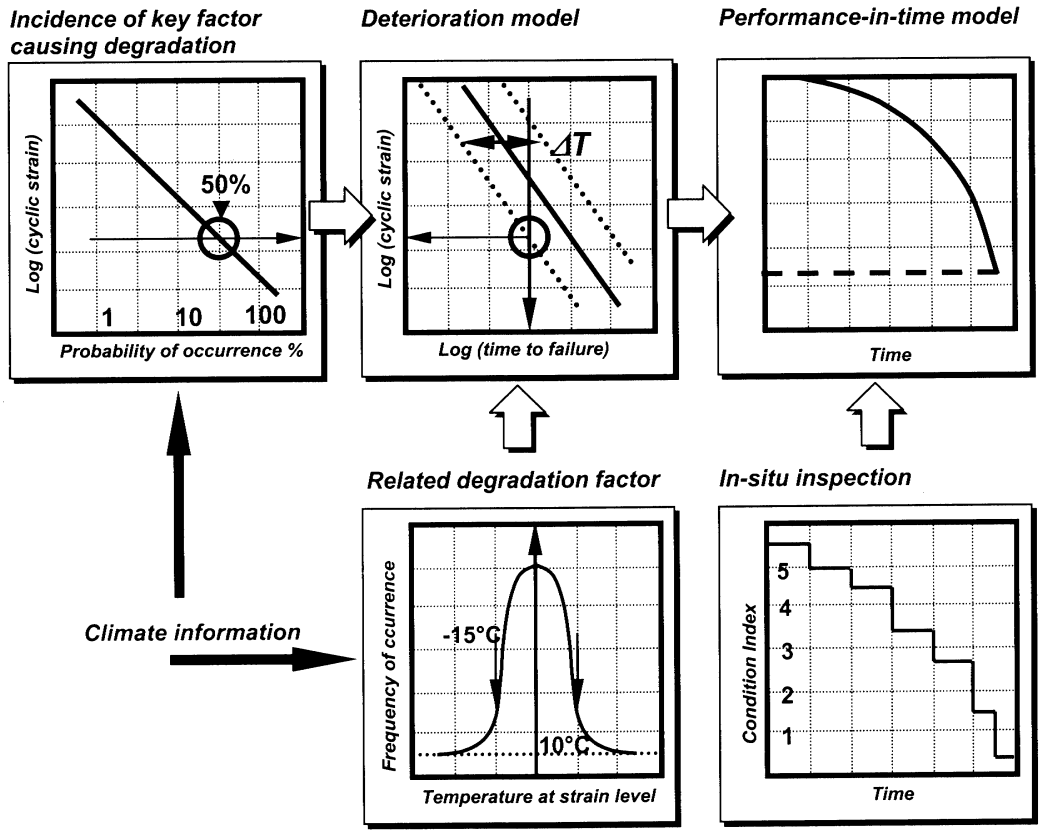


**Figure 1 - Approach to service–life prediction showing relation between degradation models and performance in time functions as adapted from RILEM [Masters and Brandt 1989] and ASTM [1996]**

Short-term tests are based on an understanding of in-use conditions and as well, knowledge of the most significant factors causing degradation. Long-term exposure tests, on the other hand, are typically undertaken in field or outdoor conditions in which climatic and related environmental effects can only be monitored but not controlled, as expected when undertaking studies in a laboratory setting. In these instances, evaluation of the long-term degradation effects on products may be determined from inspection of products used in buildings or from field trials on joints of test buildings or outdoor exposure rigs that simulate joint movement of real buildings.

Degradation models for sealants materials based on accelerated tests have either focused on the deterioration induced by rupture in the bulk of the sealant, as proposed by Lacasse et al. [1995, 1996, 1998] or the loss in adhesion to an aluminium substrate, as advanced by Shephard [1995]. Studies have also been conducted by Lacasse et al. [1994] to determine the fatigue characteristics and related mechanism of deterioration of silicone sealant in relation to fundamental studies on fatigue of rubber compounds. However none of the results from these models have used to investigate the likely behaviour of products in different climates nor been used to help estimate service life of jointing systems.

Work undertaken by Shephard [1995] was used to simulate crack growth between a one-part acetoxy-cured silicone sealant and aluminum substrate of a butt-jointed system in both Wittman, AZ and Miami, FL. The rate of crack growth along the joint interface, $\dot{a}$, is given in the equation below from which the rates of growth at different temperatures and relative humidities could be estimated.

$$\dot{a} = \left(\frac{G}{k}\right)^{1/n} \bullet \frac{1}{a_T \bullet a_{RH}}$$

in which:

$\dot{a}$ = Crack growth rate (m/s)

G = Strain energy release rate = $\int_{X_0}^{X} (mx + b)dx$

k = Constant = 1161.7

n = Constant = 0.184

$a_T$ = Temperature dependent shift factor (log $a_T$ = 6841T-20.81)

$a_{RH}$ = Relative humidity (RH) dependent shift factor (log $a_{RH}$ = 9.253 – 0.266RH + 0.0016$RH^2$)

x = Displacement of joint = 1.7•a•? T ; a = coefficient of thermal linear expansion, aluminium

m = 236040T + 1.07 x $10^8$

b = -437T + 24827

This relationship was shown to be valid for temperatures ranging between 5 and 90°C and relative humidity varying between 37 and 85%. The critical crack size for joint failure to occur was not established, however, the contrasting rates of crack growth and the climatic conditions necessary for growth to occur highlight the significance of climatic variables in establishing damage patterns. Shephard [1995] noted that the speed of crack growth greatly increased when the temperature fell below sealant application temperature and the relative humidity remained above 35%. It was suggested that climates that have wide annual changes in temperature and maintain moderate levels of humidity during cold months are likely to have the largest crack growth.

In this paper the crack growth model developed by Shepherd [1995] will be used as a basis to assess the relative importance of climatic factors to influence crack growth in particular climates located in Phoenix, Miami, Singapore, Ottawa and Winnipeg. The effect of other variables will be determined including that of installation temperature and change of modulus over time. Finally, on the basis of this model, estimates of time to failure are made as it relates to specific failure criteria. These results are limited by and relate exclusively to a given joint length, configuration and size and the physical properties of a “model unoptimized” silicone sealant product adhered to an aluminium substrate.

## 2 APPLICATION OF A CRACK GROWTH MODEL

The crack growth model requires information regarding the installation temperature, and ambient climatic conditions at a given location including temperature and relative humidity. The model was applied to various climatic conditions as described in detail below. The assumptions regarding joint details, in particular the joint type, size and configuration and panel material are required to calculate the expected movement due to thermal effects and the strain energy induced in the sealant upon movement of the joint.

### 2.1 Quantifying in-service conditions – climatic effects

Although the key environmental factors causing degradation in sealant products are well known [Wolf 2000a, b] (i.e. spectral radiation; moisture; temperature; joint movements) the vital item required to help insure the usefulness of prediction (estimation) models is quantifying the intensities of these climatic factors and their likelihood of occurrence

For the purposes of this study, simulations were carried out for the specific locations shown in Table 1. The locations were chosen such that the results from simulation could readily be compared in terms of contrasting climate variables. Specifically, hot-dry climates such as those of Wittman or Phoenix could be compared to the hot-wet climates of either Miami or Singapore, or indeed the cold-wet or cold-dry climates of Ottawa and Winnipeg respectively. The climate classifications were taken from Russo [1971]. The climatic information was obtained from Environment Canada, for Ottawa and Winnipeg, NOAA (National Oceanic and Atmospheric Administration) for Wittmann, Miami and Pheonix, and the WMO, for Singapore. The information consisted of hourly data for a given location over a specific year. Examples of the data are plotted in Figures 2 and 3 below. Hourly temperatures are shown Figure 2a for Miami, FL (1994), whereas the corresponding hourly relative humidity for the same location is given in Figure 2b. Similarly, hourly temperatures and relative humidities for Phoenix, AZ (1994) are provided in Figures 3a and 3b respectively. The values shown in Table 1 for the various climate variables were derived from the hourly data. The extent to which these are close to climate normals has not been provided.

**Table 1 – Climatic profiles for different locations**

| Location | Climate* | Average annual climate variables | | | | |
|---|---|---|---|---|---|---|
| | | Avg. T (°C) | Avg. Min. T (°C) | Avg. Max. T (°C) | Avg. % RH | Avg. rain (mm) |
| Wittman (AZ) | Hot-Dry | 20.6 | 12.2 | 28.9 | N/A | 230 |
| Phoenix (AZ) | Hot-Dry | 22.5 | 15.2 | 29.8 | 36 | 194 |
| Miami (FL) | Hot-Wet | 24.4 | 20.6 | 28.2 | 73 | 1420 |
| Singapore (SG) | Hot-Wet | 27.1 | 23.4 | 30.7 | 83 | 2413 |
| Ottawa (ON) | Cold-Wet | 6.0 | 1.2 | 10.7 | 69 | 700 |
| Winnipeg (MN) | Cold-Dry | 2.4 | -3.4 | 8.1 | 72 | 404 |

* As classified in Russo [1971] Hot: Average annual T > 15 °C; Cold: Average annual T < 15 °C; Wet: Average annual rainfall > 500-mm; Dry: Average annual rainfall < 500-mm

In comparing the hourly temperatures profiles of Miami and Phoenix it can be seen (Figures 2a; 3a) that the variation in temperatures over the year is particularly more pronounced in Phoenix even though the difference in average annual temperatures is less than 2 °C. The contrast between 'wet' and 'dry' climates is evident from comparison of the relative humidity profiles (Figures 2b; 3b) and values of annual rainfall (1420-mm vs. 194-mm) of either climate.

## 2.2 Assumptions concerning the joint configuration and product

The fictitious joint is a 2-m long vertical butt joint located on an exterior wall and sheltered from direct sunlight and rain. The assumption of sheltering precludes the need to correct for surface temperatures of the sealant given ambient conditions. In other words, the increase in temperature of the sealant due to the effects of exposure to direct solar radiation need not be considered. The butt joint is located between aluminium panels of 1.7-m width. The joint is 12.7-m wide by 12.6-mm deep. The thermally induced joint movement, d, is simply calculated assuming that the panels are fixed to the structure at their mid-points; hence d = 1.7•a•? T, where a is the coefficient of linear thermal expansion of aluminium (23.2 x $10^{-6}$ mm/mm °C) and ? T is the temperature differential causing movement.

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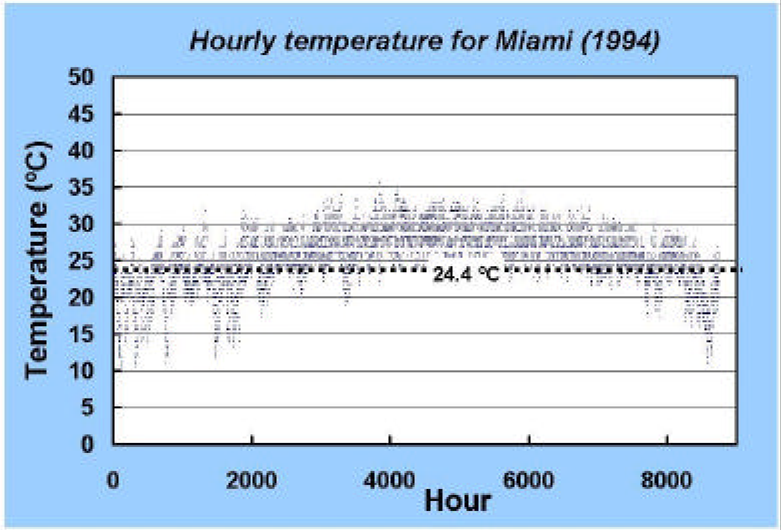


Figure 2a – Hourly temperatures in Miami (1994)

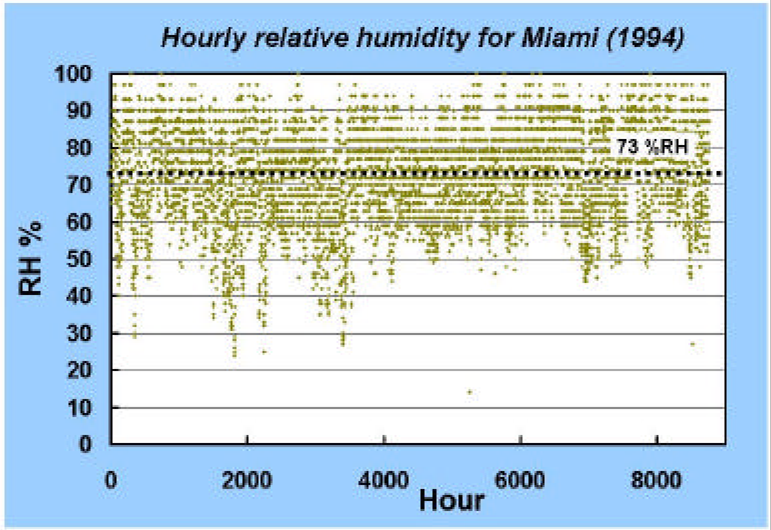


Figure 2b – Hourly relative humidity in Miami (1994)

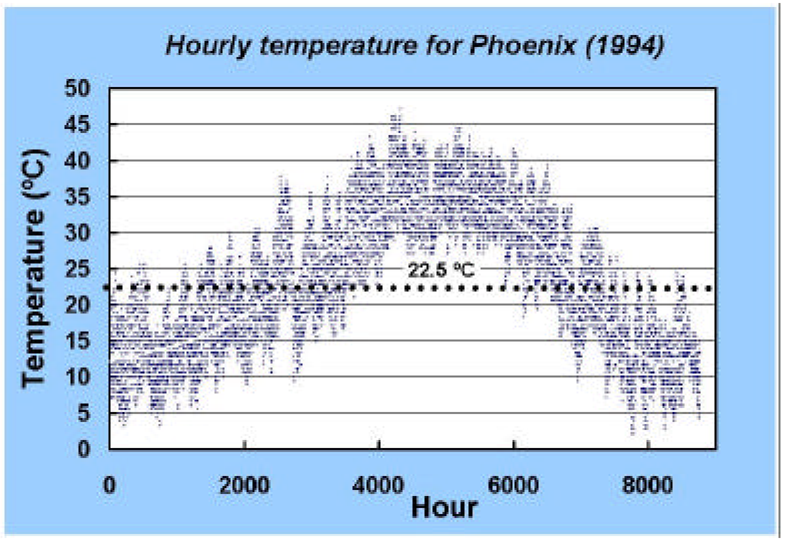


Figure 3a – Hourly temperatures in Phoenix (1994)

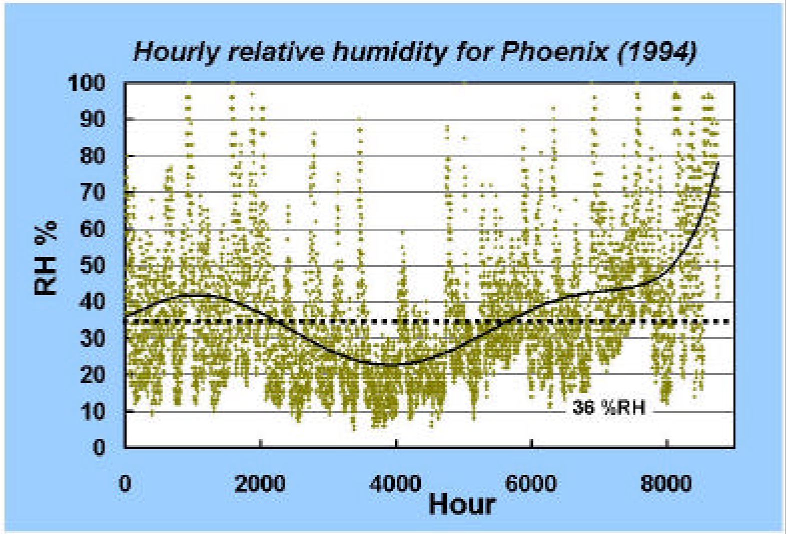


Figure 3b – Hourly relative humidity in Phoenix (1994)

The sealant considered was an acetoxy-based one-part moisture curing silicone product that was assumed to cure at 30°C. Strain energy calculations were not taken into consideration when ambient local temperatures exceeded the installation temperature give that in these conditions, the joint is in compression and crack growth is assumed to occur only when in tension. This is a reasonable assumption so long as the compressive strains are less than 50% or the sealant cross section is hourglass shaped.

### 2.3 Variables investigated

Four items were investigated using the crack growth model to simulate sealant deterioration: the nature of crack growth in nominally similar as well as dissimilar climates; the significance of the installation temperature on crack growth development in the various climates investigated; the possibility of estimating time to failure for given failure criteria; and, emulating the effects of ageing through increases in modulus.

## 3 RESULTS

### 3.1 Comparison of crack growth in different climates

Depicted in Figure 4 is the crack length development over a period of one year (8760 hours) for a joint product located in Wittman (AZ) and Miami (FL). The assumed installation temperature was 30°C and is above the average annual temperatures of either location. This implies that crack growth is arrested (horizontal portion of plot) for most of the year and growth is thus limited to those months in which hourly temperatures are typically well below the 30°C range. Note as well, that the rates of growth either preceding or following that portion showing arrested growth are nominally the same.

The model also suggests that the expected annual crack growth in Miami may be up to ca. 5 times less significant than that of Wittman. The differences are almost entirely attributable to the magnitude of differences in temperature between either the climate during the ‘colder’ months of the year.

The expected consequences of these simulated phenomena in regard to longevity of the seal are apparent although no pronouncements can be made unless criteria for failure or loss in performance are first established.

The annual crack growth profiles derived from model simulations for locations having hot-wet climates is provided in Figure 5 from simulations of crack growth in Singapore and Miami. It is evident that the relative overall annual crack growth in Miami (ca. 0.425-mm) is significantly greater than that of Singapore (ca. 0.003-mm Singapore), about 2 orders of magnitude difference.

Whereas the growth in Miami appears to be dominated by seasonal effects with growth occurring primarily in the colder months, growth in Singapore is reasonably steady over the year (see inset) indicating that diurnal variations in both temperature and relative humidity are causing the growth effects.

Although both climates are classified as being ‘hot-wet’ there is nonetheless an evident difference in the manner in which the model responds to climate loads. When considering that the difference between the average annual temperature in Singapore (27.1°C; 83% RH) and Miami (24.4°C; 73% RH) is 2.7°C and that the difference in terms of relative humidity is ca. 10% the model appears reasonably useful in being able to readily discern between nominally similar climates. The spread between the average maximum temperature in Singapore is 7.4°C about the same as that which occurs in Miami (7.6°C)

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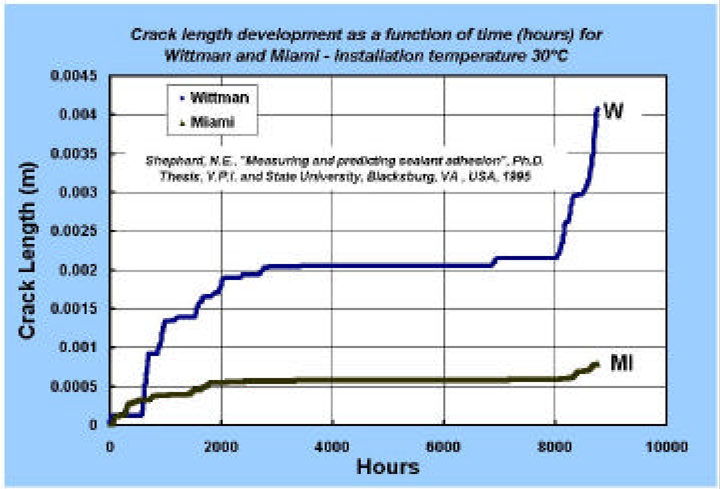


Figure 4 – Simulated hourly crack growth of product over 1 year (8760 hours) in Wittman (hot-dry) and Miami (hot-wet) climates installed at 30 °C.

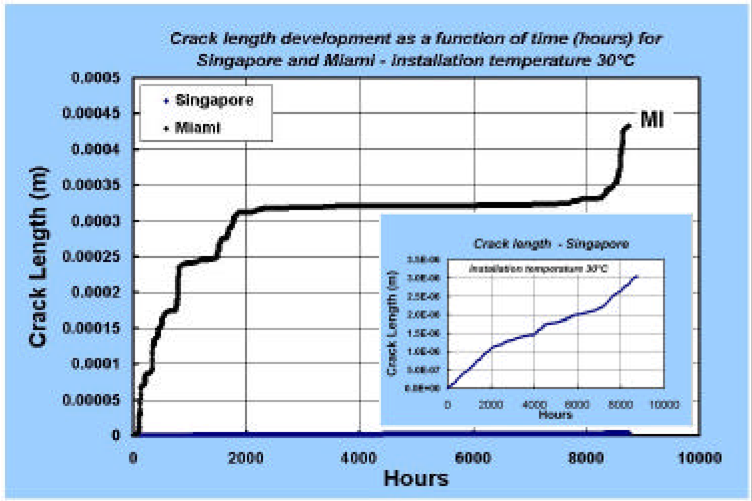


Figure 5 – Simulated hourly crack growth of product over 1 year (8760 hours) in Singapore and Miami climates installed at 30 °C. Both climates are 'hot-wet'. Inset shows hourly crack growth in Singapore over 1 year.

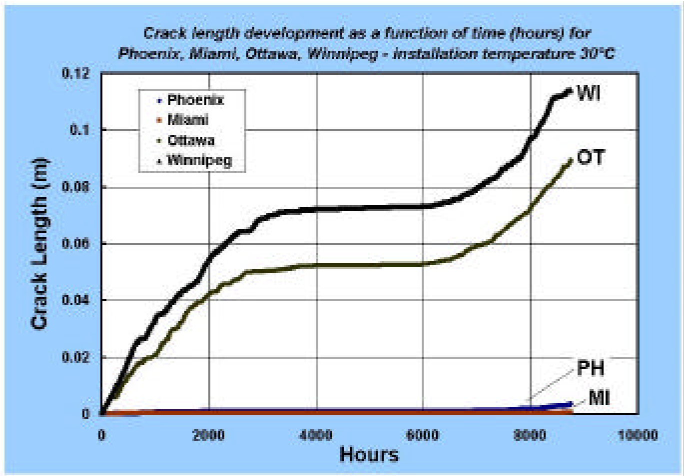


Figure 6 – Simulated hourly crack growth of product over 1 year (8760 hours) in Phoenix (hot-dry), Miami (hot-wet), Ottawa (cold-wet) and Winnipeg (cold-dry) climates installed at 30 °C.

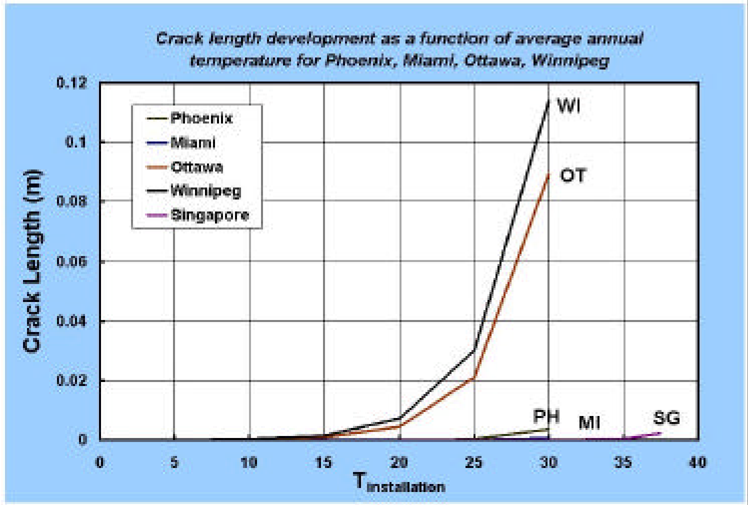


Figure 7 – Simulated total annual crack length development for various locations in relation to installation temperature

Shown in Figure 6 are the simulated crack growth profiles of jointing systems located in hot-wet (Miami), hot-dry (Phoenix), cold-wet (Ottawa) and cold-dry (Winnipeg) climates. Phoenix is physically in close proximity to Wittman hence the climate variables are similar and the comparison between Phoenix and Miami is similar to that shown in Figure 4 for Wittman and Miami. Shown in this figure are simulations undertaken for Ottawa and Winnipeg, both characterized as generally cold climates (Table 1), the climate in Winnipeg being cooler (average annual temperature - 2.4°C) than that of Ottawa (6.0°C) and also dryer (404-mm avg. annual rain vs. 700-mm).

The crack growth profiles for these two locations offer significantly greater annual crack growth in comparison to either of the values obtained for Phoenix and Miami; ca. 90-mm and 115-mm for Ottawa and Winnipeg respectively as compared to 0.425-mm for Miami. The period of growth in the cooler months is both more significant and longer lasting. More significant because the growth rate during the colder months is greater and longer lasting, evidently because the period of 'dormancy', (period during which growth rate is comparatively less) is shorter as compared to that of, e.g., Miami.

### 3.2 Significance of installation temperature in the development of crack growth

The rate of simulated crack growth is largely dependent on the temperature of installation. This occurs because it is assumed that when ambient temperatures fall below the installation temperature the joint widens in response to contractions of adjacent panels. Hence the product is in tension and this effect is increasingly pronounced in relation to the temperature difference that exists at installation and ambient conditions. Clearly then, if the jointing product is installed at 30°C, the most pronounced effects will occur where there exists the greatest difference between the average annual temperatures and installation temperature as is the case for both Ottawa and Winnipeg.

This is illustrated in Figure 7 that shows the significance of the installation temperature on the simulated annual crack length development for the five locations. Total crack lengths after one simulated year are plotted as a function of installation temperature for Phoenix, Miami, Ottawa, Winnipeg, and Singapore. It shows that the significant crack lengths developed in both Ottawa and Winnipeg as compared to the other locations (Phoenix, Miami, Singapore) is directly attributable to the temperature of installation. Simply put, installation at temperatures higher than the average annual temperature for a given location, as is the case for Winnipeg and Ottawa, produces greater simulated annual crack length development as compared to those locations where the installation temperature is closer to that of the average annual temperature (i.e. Phoenix, Miami and Singapore). It also shows that the annual crack growth diminished quite significantly for both Winnipeg and Ottawa as the installation temperature approaches the average annual temperature at these locations. This observation is in keeping with good installation practice that suggests undertaking sealing operations as close as possible to the average annual temperature such that the joint can thereafter operate equally in compression as in tension. Figure 7 also illustrates that there is an increased risk of failure when joints are installed at temperatures well in excess of their average annual value.

If products were installed at the average annual temperature of the location, what would be the magnitude of the annual crack development at the different locations? This is shown in Figure 8 in which the simulations were conducted for each of the locations at an installation temperature reflecting the respective average annual temperatures of the location with the exception of Winnipeg. In this instance, installation was assumed to be 5°C since this is typically the lowest permitted temperature at which products are installed.

The results indicate that the magnitude of the crack growth over the year is reduced in all cases in comparison to those values obtained when products are installed at 30°C. Values for Miami, Ottawa and Winnipeg are in the same order of magnitude, ranging from ca. 0.015-mm (Miami) to ca. 0.04-mm (Winnipeg), whereas Phoenix is shown to exhibit crack growth of up to ca. 0.11-mm. Hence of the locations investigated, interestingly Phoenix is comparatively the most severe climate, having about 2-3 times more growth over the year (0.11-mm) as compared to, e.g. Winnipeg (ca. 0.04-mm).

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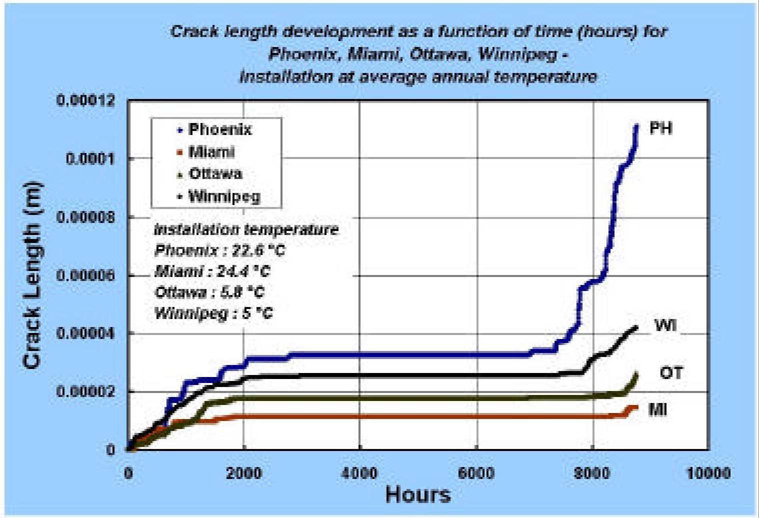


Figure 8 - Simulated hourly crack growth profiles for Phoenix, Miami, Ottawa and Winnipeg for products installed at average annual temperature

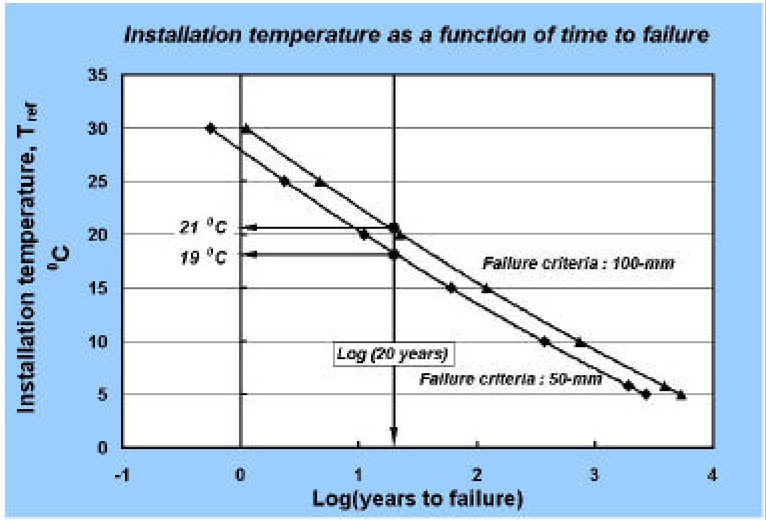


Figure 9 – Time to failure for given installation temperatures and failure criteria (50-mm and 100-mm) for a product installed in Ottawa

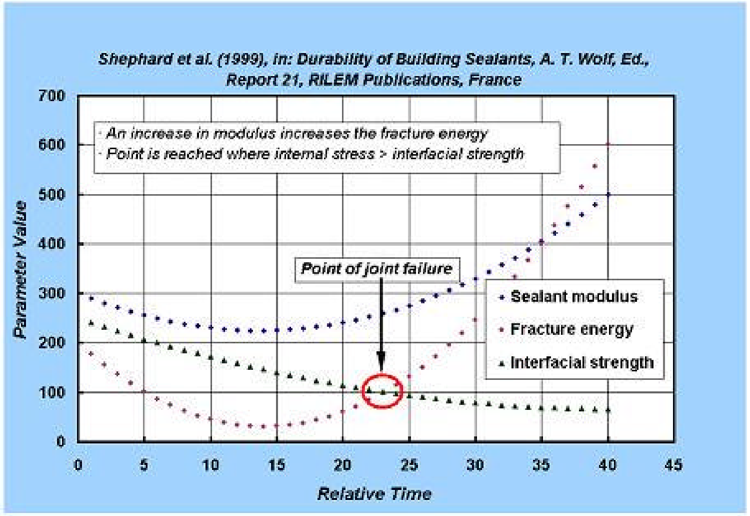


Figure 10 - Relative changes in applied fracture energy and interfacial strength [Shephard 1999]

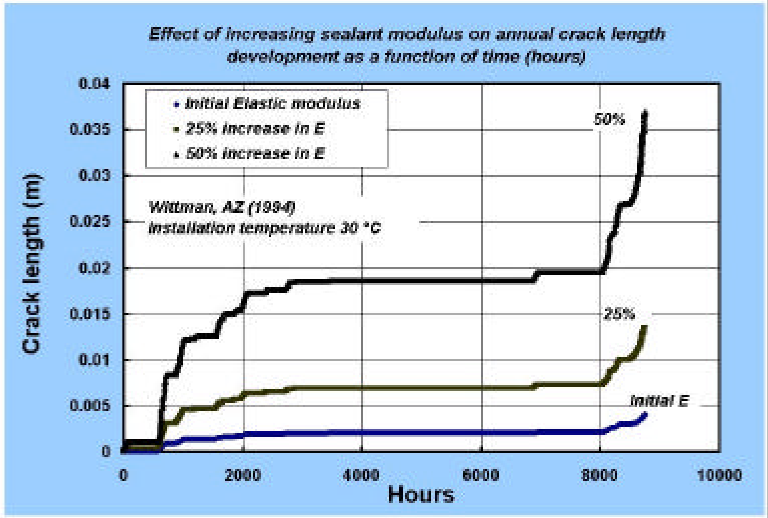


Figure 11 –Simulated hourly crack growth over a year in Wittman for a butt joint having increases in modulus of 25 and 50% [Shephard 1999]

### 3.3 Estimates for time to failure

It has been shown that significant crack growth can develop over a period of a year if installation is undertaken at temperatures well in excess of the average annual value (Figure 7). As well, these effects are seen to diminish significantly if the installation temperature is close to that of the average annual value. For example, in Ottawa the annual value for crack growth derived from simulation was shown to be 0.03-mm. Estimates for the service life of a joint can be made on the basis of this crack growth model provided criteria for loss in performance is established. For example, the service-life of a 2-m joint may be reached when along this length a crack of, e.g., 50 or 100-mm is detected. Given that the average annual crack length can be determined from simulation, a projection can then be made as to when a certain length of crack will appear, assuming that in each subsequent year, climatic conditions are essentially the same.

The results of this annual progression for a joint installed in Ottawa (Avg. An. T = 6.0°C) at various temperatures are provided in Figure 9. Two plots are shown that provide estimates of service life in terms of the log (years to failure) where the failure criteria is the development of either a 50 or
100-mm length of crack. A service-life of 20 years is highlighted because this value is one that is often ascribed to high-performance sealant products.

The results from simulation suggest that if the products are installed at the average annual temperature of a given location (in this case Ottawa), they may last indefinitely however, if installed at higher temperatures, e.g. ca. 20°C, the service life is likely to diminish.

### 3.4 Effect of change in modulus over time

Necessarily, a number of different factors other than crack development contribute to the deterioration of jointing seals. Products typically harden as they age as manifested by the increase in modulus over time. As shown in Figure 10 [Shephard 1999], increases in modulus directly affect the fracture energy. Once the modulus starts to increase there is then a corresponding increase in the fracture energy to which the interfacial strength of the adhesive bonds is likewise affected. Joint failure essentially occurs when internal stresses, brought about by higher fracture energies, exceed the interfacial strength of the adhesive bond.

To illustrate this aging effect in an indirect manner, aging is reproduced by increases in modulus and thereafter, crack growth was simulated on a joint installed at 30°C in Wittman, as shown in Figure 11 [Shephard 1999]. The profiles show crack growth of the product having the initial elastic modulus increased by 25 and 50 % respectively; increases in crack growth evidently occur over the year due to the increases in modulus. It appears that growth profiles nominally increase from 0.4-mm to 14-mm for a 25% increase in modulus to 38-mm for a 50% increase. These results simply emphasize the significance of modulus changes to that of crack growth development.

## 4 SUMMARY

A crack growth model has been used to study the development of crack profiles for a specific joint type and configuration and sealant product in different climates. It is particularly sensitive to variations in temperature and less so to changes in relative humidity. As such it is able to discern the apparent crack growth development of nominally similar climates and as swell, to readily differentiate between the effects brought about in dissimilar climates. It can be used to assess the relative significance of installation temperature at given locations and as well could form the basis for the development of a service life model provided results from the simulations are compared to those derived from controlled field (outdoor) tests. As well, the method can be applied to other sealant products and sealant–substrate combinations to determine their comparative crack growth characteristics.

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# Aspects Of Durability Of Self Compacting Concrete

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**Summary**: **Self-compacting concrete (SCC) avoids the need for compaction when placing fresh concrete. This saves time, reduces overall cost, improves working environment and opens the way for automation of the concrete construction. Since the development of SCC over 10 years ago, extensive research has been carried out on the mix design, placing method, fresh properties and strength of various SCC mixes. Research on durability of SCC, however, has been very limited, particularly on durability performance of different types of SCC mixes and its comparison with that of conventional vibrated concrete of a similar strength.**

**In this study, gas permeability, capillary water absorption and chloride diffusivity, which are important indicators for concrete durability performance, of various types of SCC and conventional vibrated reference concrete mixes were assessed and compared. The results of a limited study on micro-properties of interfacial transition zone (ITZ) around steel reinforcement were also reported. The results appeared to indicate that SCC mixes had significantly lower permeability and capillary absorption, similar chloride diffusivity compared to the vibrated reference concrete of the same strength grade. The durability performance of SCC mixes was also affected by the use of filler materials. The durability performance of the SCC mix containing no additional filler but using a viscosity agent was found to be inferior to those containing additional fillers.**

**Keywords. Self-compacting concrete, Durability, Transport properties, Micro-properties, Gas permeability, Chloride diffusivity.**

## 1 INTRODUCTION

The introduction of self-compacting concrete (SCC) to the construction industry has resulted in a major change in concrete construction process, with large benefits being made not only in productivity but to the working environment on site. Since its development in the late eighties there has been a large amounts of research carried out on the mix design, placing methods, fresh properties and strength of various SCC mixes (Ozawa & Ouchi 1998, Skarendahl & Petersson 1999, Gibbs, Bartos *et al*. 2000), but only a very limited amounts on its durability performance (Gram & Piiparinen 1999, Tang *et al*. 1999).

SCC mixes always contain a powerful superplasticizer and often use a large quantity of powder materials and/or viscosity-modifying admixtures. The superplasticizer is necessary for producing a highly fluid concrete mix (low yield value) while the powder materials or viscosity agents are required to maintain sufficient viscosity of the mix, hence reducing bleeding and segregation/settlement. The powder materials used often include limestone powder, pulverised fuel ash (PFA), granulated ground blastfurnace slag, etc. Furthermore, coarse aggregate content is much lower in SCC mixes than in traditional vibrated concrete mixes to reduce the risk of blocking of concrete flow by congested reinforcement and narrow openings in the formwork. Due to such significant differences in the mix proportions and also in placing and compaction processes between the SCC and traditional vibrated mix, it is inadequate to assume that SCC would have the same durability characteristics as traditional concrete if their strength grade were similar. In view of its growing popularity of use in structures that require a high standard of durability, knowledge of the durability performance of SCC mixes is urgently needed. This study was carried out to systematically investigate aspects of durability of different types of SCC mixes in comparison with vibrated reference normal concretes of a same strength grade.

## 2 EXPERIMENTAL DETAILS

### 2.1 Concrete mixes used

To cover the range of different mix variations, three SCC mixes were designed, namely, one using limestone powder and one using PFA as additional powders and another one using no additional powder but a viscosity agent - Welan gum. The SCC designs were determined by trial mixes or based on previous work for the European Brite EuRam project: "Rational production and improved working environment through using self-compacting concrete, 1997 - 2000" (Gibbs, Bartos *et al*. 2000). A special superplasticising admixture, Viscocrete 2, provided by Sika was used in SCC mixes to achieve a slump flow value of 600 - 650 mm. Two reference mixes were selected, one using Portland cement only, the other using Portland cement and PFA as the binder material. The reference mixes with a medium workability (slump = 50 - 70 mm) were determined using the DOE method (Teychenne *et al*. 1988) and verified by trial mixes. All the SCC and reference mixes were designed for characteristic cube strength of 40 MPa (C40). Details of the concrete mixes used and their basic properties are given in Table 1.

**Table 1. Mix designs and basic properties of SCC and reference concretes**

| **Mix proportions, kg/m³** | REF 1 | REF 2 | SCC 1 | SCC 2 | SCC 3 |
|---|---|---|---|---|---|
| Granite, 20 - 5 mm | 1105 | 1045 | 770 | 770 | 770 |
| Natural sand, zone M | 715 | 695 | 875 | 875 | 990 |
| Portland cement, 42.5 | 340 | 280 | 285 | 335 | 360 |
| Fine limestone powder | - | - | 265 | - | - |
| PFA, BS 3892 Part 1 | - | 120 | - | 145 | - |
| Viscosity agent (Welan gum) | - | - | - | - | 0.17 |
| Superplasticizer (Viscocrete 2) | - | - | 5.0 | 4.8 | 7.2 |
| Free water | 195 | 190 | 180 | 195 | 210 |
| Volume of paste, litre/m³ | 303 | 335 | 369 | 369 | 324 |
| **Basic properties** | | | | | |
| Slump, mm | 65 | 50 | - | - | - |
| Slump flow, mm | - | - | 620 | 630 | 600 |
| 7-day compressive strength, MPa | 29.5 | 27.5 | 40.5 | 33.0 | 32.5 |
| 28-day compressive strength, MPa | 45.5 | 43.0 | 51.0 | 50.0 | 41.5 |

The results in Table 1 show that all concrete mixes achieved the required strength grade C40 and the required workability. The slump flow values for the SCC mixes were achieved by adjusting the dosage of the superplasticizer, and all the mixes showed good resistance to segregation.

### 2.2 Test procedures

Standard testing for compressive strength was carried out at the ages of 7 and 28 days. Oxygen permeability test measurements, capillary water absorption test and chloride diffusivity measurements were made to provide comparisons of durability performance between the SCC and the reference mixes, as well as among the different SCC mixes. These followed the methods described by Kollek (1989), RILEM TC 166 (1999) and Tang & Nilsson (1992) respectively. The specimens used for the permeability and the capillary absorption tests were pre-conditioned through oven-drying, while the specimens for the chloride diffusivity test were pre-conditioned through vacuum-drying and saturation with $Ca(OH)_2$ solution. The pre-conditioning of specimens were started at the age of 7 days in order to simulate the usual practical site conditions.

Limited supplemental tests based on water absorption and micro-indentation methods were also carried out to determine the water accessible porosity (Tang, 1999) and micro-properties of the interfacial transition zone between steel reinforcement and concrete (Zhu & Bartos 2000).

## 3 RESULTS AND DISCUSSIONS

### 3.1 Coefficient of oxygen permeability

The coefficient of permeability is a materials characteristic describing the permeation (or flow) of fluids through a porous material due to a pressure head. The permeability of the concrete specimens being investigated in this study was determined using the CEMBUREAU permeameter, as shown in Fig. 1. Oxygen was used as the permeating medium and a set of constant pressure heads 0.5 to 2.5 bar was applied to the test specimen. The test procedure entailed measuring the flow rate of gas through a concrete specimen at steady state under the pressure head. This allows the permeability coefficient of the tested concrete to be determined.

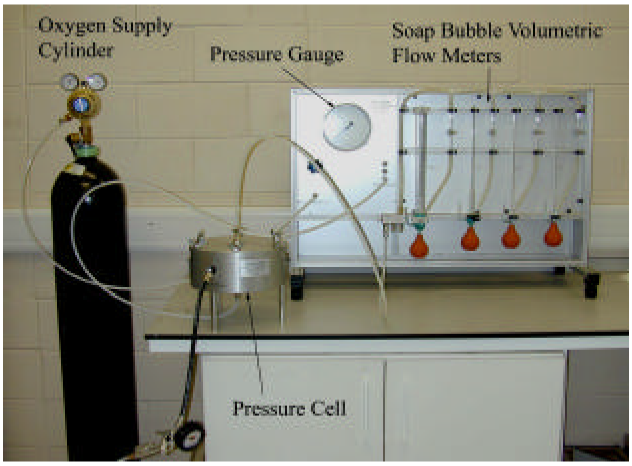



**Figure 1. Oxygen permeability test apparatus used**

Two replicate samples from each of the concrete mixes, preconditioned in a 60 $^{o}$C oven for two weeks, were tested at up to five test pressures. The results of permeability coefficient obtained were analysed and the average data and their standard deviations are given in Table 2.

**Table 2. Results of coefficient of oxygen permeability**

| Coefficients of oxygen permeability, m² x $10^{-17}$ | REF 1 | REF 2 | SCC 1 | SCC 2 | SCC 3 |
|---|---|---|---|---|---|
| Average result | 12.8 | 13.9 | 5.5 | 4.1 | 8.2 |
| Standard deviation | 0.5 | 1.1 | 0.2 | 0.2 | 1.5 |

The results in Table 2 clearly indicate that all three different SCC mixes had significantly lower oxygen permeability coefficient than the traditional vibrated reference concrete mixes. Particularly, for SCC mixes using PFA and limestone powder (i.e. SCC2 and SCC1 respecitively), the coefficient of permeability is only 30 - 40% of the level for the reference concrete mixes. Among the three different SCC mixes, the SCC3 which contained no additional powder but used a viscosity agent was found to have higher permeability coefficient than the other SCC mixes which contained additional powder.

### 3.2 Sorptivity - capillary water absorption

The transport properties of the near-surface concrete are of major importance when dealing with the durability of reinforced concrete. This is because most of the deterioration processes affecting structures involve the transport of aggressive fluids into the concrete. These processes include freeze/thaw attack, internal expansion due to chemical reactions and the ingress of dissolved specimens, particularly chlorides and sulphates. Capillary absorption is one of the main mechanisms that control the transport characteristic of the near-surface concrete.

In this study, the sorptivity - rate of capillary water absorption through concrete surface, was determined. The specimens used (150 mm cubes) were pre-conditioned through drying in oven at 105±5 $^{o}$C to constant weight. After cooling the specimens were prepared and tested according to the procedures described in the RILEM TC 166 (1999) recommendation. The uptake of water by capillary absorption was measured through the weight gain of the specimen at the set time intervals of 10 minutes, 30 minutes, 1 hour, 4 hours and 24 hours of concrete surface in contact with water. The uptake of water per unit area of concrete surface I, followed a linear relationship with the time for the suction periods t, namely, $I = S\ t^{0.5}$, where S is usually called sorptivity. The sorptivity can be obtained from the slope of the I vs. $t^{0.5}$ plot. The results of sorptivity for the SCC and the reference mixes are presented in Fig. 2.

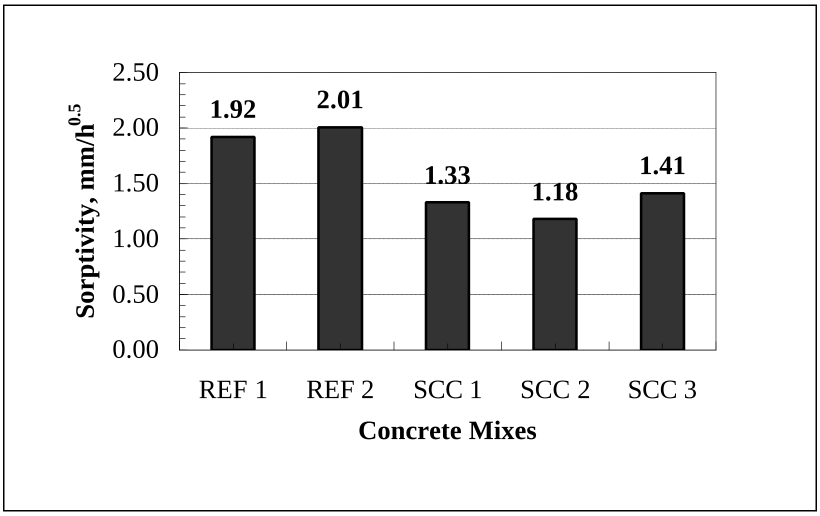



**Figure 2. Results of capillary absoption of water through concrete surface**

The results in Fig. 2 clearly indicate that the sorptivity was considerably lower for all the SCC mixes than for the traditional vibrated reference mixes. Such results would suggest that the near-surface concrete was denser and more resistant to ingress of fluid in the SCC mixes than in the corresponding reference mixes. SCC 2, which used PFA as additional powder, showed the lowest sorptivity value of all the mixes tested. The SCC mix with the highest sorptivity value was SCC 3 which used no additional powder but the viscosity agent Welan gum. The results confirm the previous findings that SCC mixes had lower sorptivity values than those of traditional vibrated concrete mixes of the same strength grade (Gibbs, Bartos *et al*. 2000). It is also interesting to note that the order of relative magnitude of sorptivity of all the mixes appeared to be in consistent with the order of results for the oxygen permeability coefficient.

### 3.3 Chloride diffusivity

Chloride ingress into concrete is one of the most common causes of durability problems, particularly corrosion of reinforcement in structural concrete. The ingress of chloide into concrete may be by permeation and capillary absorption of chloride-containing solutions or by diffusion of chloride ions through saturated internal network of pores in concrete. The theorectical description of the chloride ion ingress due to a diffusion process is basically done according to Fick's second low of diffusion.

In this study, the determination of chloride diffusivity of concrete specimens was carried out using the CTH rapid test (Tang & Nilsson, 1992). The test arrangement for the CTH rapid method is shown in Fig. 3. This is an electrically accelerated method providing rapid measurement of diffusivity (i.e. coefficient of migration) of chloride ions in concrete. This test involves placing a 50 mm thick preconditioned saturated concrete specimen in a test cell, with its test face in contact with a chloride solution and imposing an external potential of 10-30 volts across the specimen to force the chloride ion migrating into the specimen. After a duration of up to 24 hours, the fresh specimen is axially split in two and the average depth of penetration, shown by the application of a silver nitrate solution, is measured. This average depth can be then used to calculate the coefficient of chloride migration.

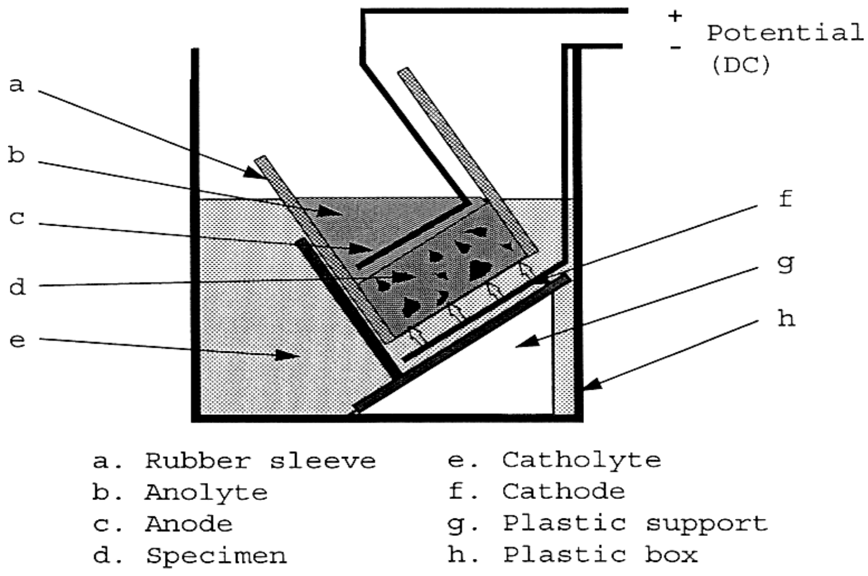



**Figure 3. Test arrangement of the CTH method (Tang & Nilsson, 1992)**

**Table 3. Results of coefficient of chloride migration and water accessible porosity**

| Coefficients of chloride migration, $m^2/s \times 10^{-12}$ | REF 1 | REF 2 | SCC 1 | SCC 2 | SCC 3 |
|---|---|---|---|---|---|
| Average result | 56.8 | 14.9 | 52.5 | 9.6 | 66.1 |
| Standard deviation | 10.4 | 0.6 | 2.5 | 1.7 | 2.9 |
| Water accessible porosity, % | 13.7 | 12.9 | 13.4 | 11.9 | 15.3 |

The results of the coefficient of chloride migration obtained were analysed and the average data and their standard deviations, together with results of water accessible porosity are given in Table 3. These test results clearly indicate that the resistance to penetration was very much affected by the type of additional powders used in the concrete. This can be seen in the very low migration coefficient values achieved by the reference mix, REF 2, and the self-compacting concrete, SCC 2, which both used PFA. This is in agreement with previous findings that PFA concrete had a higher resistance to chloride penetration than OPC concrete with similar compressive strengths (Dhir, *et al*. 1990). The self-compacting concrete mix using the fine limestone powder, SCC 1, and the remaining traditional vibrated reference mix, REF 1, showed similar values of chloride migration coefficient. SCC 3, which contained no additional filler but a viscosity agent had relatively the highest chloride migration coefficient and therefore had the least resistance to chloride penetration out of all the mixes tested.

The results in Table 3 also appeared to indicate that the order of relative magnitude of chloride diffusivity of all the mixes was in consistent with the order of results for the water accessible porosity.

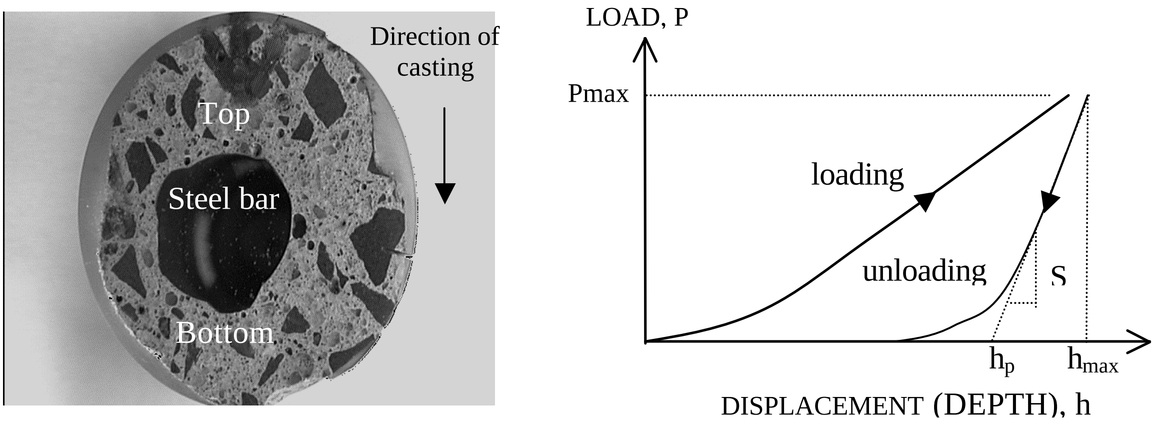
This may be explained by the fact that water plays an essential role in the transport process. Diffusion or migration of chloride ions in concrete is only possible through continuous water or liquid path in the capillary pore or micro-crack system within concrete. Thus, the significant reduction of chloride diffusivity due to the incorporation of PFA may be the results of more tortuous or blocked transport path since the spherical PFA particles could improve the particle packing density both in the matrix and in the interfacial zone around coarse aggregate.

### 3.4 Micro-properties of interfacial transition zone around steel reinforcement

The interfacial transition zone (ITZ) between cement paste and reinforcement (i.e. aggregates and steel bars) has been recognised as being the 'critical and weak link' in cement composites and structural concrete, and having a considerable influence on the engineering and durability properties. In order to assess the impact of the use of SCC on bond and durability performance, the micro-properties of the ITZ around steel reinforcement was studied using a novel depth-sensing micro-indentation test (Zhu & Bartos, 2000).

Two concrete mixes with a characteristic cube strength of 35 MPa (C35) were examined: one SCC mix (i.e. C35-SCC) containing fine limestone powder, with mix proportions being similar to those of SCC 1, and the other a reference mix (i.e. C35-REF) with mix proportions being similar to those of REF 1. The specimens for the micro-indentation test, as shown in

Fig. 4a, were extracted and prepared from practical full-scale reinforced concrete elements. An unique nano-technology based apparatus which continuously measures indentation load and displacement during a micro-indentation test was used to study the micro-properties of the ITZ on the top and bottom side of the reinforcement. A typical outcome of such a test is an indentation load-displacement (or depth) hysteresis curve as shown in Fig. 4b. Determination of the elastic recovery by analysing the unloading data leads to a solution for calculation of elastic modulus E and also microhardness H of the test area. Details of the theorectical background and methodology have been reviewed and presented by Oliver & Pharr (1992), Doerner & Nix (1986) and Trtik & Bartos (1999).




(a) (b)

**Figure 4. Micro-indentation testing of the ITZ: (a) test specimens, and**

**(b) typical indentation load vs. displacement (depth) curve (Zhu & Bartos, 2000)**

A small diamond tip (i.e. indenter probe) with a 90° geometry was used for this investigation. A pre-specified indentation depth of 10 µm was used throughout the testing and this produced an indent (or impression) of the size of about 20 µm in width. A large number of indentation tests were made in the ITZ both on the top and bottom side of the horizontal steel bar. The adjacent test points were kept 60 - 80 µm apart so to avoid possible overlapping of the areas affected. The test results were analyzed and are given in Table 4.

**Table 2. Average results of the micro-properties of the ITZ around steel bar**

| Specimens | | Elastic modulus, GPa | | Microhardness, MPa | | Estimated width of ITZ, µm |
|---|---|---|---|---|---|---|
| | | Average E | STD | Average H | STD | |
| C35-REF | Above steel bar | 14.1 | 0.9 | 320 | 38 | 45 |
| | Below steel bar | 11.7 | 3.0 | 260 | 50 | 50 |
| C35-SCC | Above steel bar | 19.5 | 1.1 | 380 | 32 | 35 |
| | Below steel bar | 18.5 | 3.3 | 326 | 93 | 35 |

The results in Table 4 appeared to indicate that the average elastic modulus and microhardness values of the ITZ were higher in the SCC mix than in the corresponding traditional vibrated reference mix. As expected, the ITZ properties below steel bar were found to be relatively weaker than those above the bar, likely to be caused by internal bleeding and settlement of the fresh concrete during the placing and compaction processes. However, the results seemed to indicate that the difference of ITZ properties between top side and bottom side of the steel bar was less pronounced for the SCC mix than for the reference mix.

## 4 CONCLUSIONS

Results of aspects of durability including oxygen permeability, capillary water absorption, chloride diffusivity and interfacial micro-properties of a range of different SCC mixes are presented in comparison with those of selected traditional vibrated reference concretes of the same strength grade of C40. Further work on the C60 grade concrete is yet to be completed and the results will be available at the time of the conference. For the range of SCC and reference mixes studied the following conclusions can be drawn:

- SCC mixes showed significantly lower values of coefficient of permeability and sorptivity of water absorption, compared to the traditional vibrated reference mixes of the same strength grade.

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- SCC mixes also showed similar chloride diffusivity to those of traditional vibrated mixes. However, the chloride diffusivity was found to be very much dependant on the types of powder used in concrete. Both the reference and SCC mixes containing PFA showed much lower values of coefficient of chloride migration than the other mixes.
- Among the three different SCC mixes, it appeared that the SCC mix containing no additional powder but using the viscosity agent to maintain stability of the fresh mix had the highest permeability, sorptivity and chloride diffusivity, thus less resistant to ingress of aggressive fluids.

The results have to be considered in the context of Self-Compacting Concrete not being another “special” type of mix with unique properties. Almost any concrete of any strength or durability can be produced in such a manner that it is self-compacting when fresh. However, SCCs typically contain much higher proportions of fines than traditional vibrated mixes, or the absence of such fines is mitigated by additional admixtures (viscosity agents). The conclusions shown apply to mixes selected as typical examples of SCC.

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# Life Cycle Assessment – A Practical Tool For Sustainable Design

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**Summary: As part of its international commitment to the global environment, the Australian Government is set to ensure that the construction industry moves towards the application of design and usage principles that will result in ecologically sustainable outcomes. While there is an increasing awareness among designers and builders of many of the environmental issues that need to be considered in the design, construction and operation of buildings, little exists to support them in their application.**

**With the overall increase in environmental consciousness, decisions regarding the selection of construction technology and construction materials can no longer being based solely on the technical and economical aspects but are becoming increasingly influenced by environmental considerations. This implies that the most favourable alternative from a balanced technical, economical and ecological perspective will be selected.**

**Whereas the technical and economical characteristics of a structure are relatively easy to quantify, assessment of the environmental impact of a building has been difficult to achieve. The lack of availability of easy to use and accessible measurement tools has led to inadequate and often misleading appraisals of a structures environmental performance.**

**In response to this, the methodology and modelling of Life Cycle Assessment (LCA) has been evolving.**

**This paper presents an overview of LCA methodology and its relevance and application in the construction industry. The paper provides a summary of the current activities in the development and use of LCA as a tool for assessing the environmental impact of buildings. It also identifies areas where further developments are required.**

**Keywords: Life-cycle Assessment, Environment, Sustainable development.**

## 1 INTRODUCTION

There is a clear and definite commitment by the Australian Government to ensure that the construction industry moves towards ecological sustainability. There is an increasing awareness among practitioners (designers and builders) that there are many environmental issues that need to be considered in the design, construction and operation of buildings to ensure that the built environment is sustainable.

With the overall increase in environmental consciousness, decisions regarding the choice of type of construction and of construction materials can no longer be made solely from technical and economic points of view but are becoming increasingly influenced by environmental aspects. The traditional criteria for selecting a particular structure type have been engineering requirements, initial and life cycle costs, experience with and availability of the materials and technology, aesthetics and the ability to build the structure under the local conditions. In the event of alternatives having identical technical performance characteristics the cost would have been, generally, the decisive factor. In contrast to this there is a call today for sustainable construction, that is resource-conscious and environmentally friendly structures. This implies that the overall most favourable alternative from the technical, economical and ecological points of view will be used.

Whereas the technical and economical characteristics of a structure are comparatively easy to quantify, an assessment from the ecological point of view is more difficult to carry out. To address the latter the concept of Life Cycle Assessment (LCA) has been evolving.

This paper presents an overview of LCA methodology and discusses its relevance and application in the construction industry.

## 2 LIFE CYCLE ASSESSMENT (LCA)

LCA is a method that systematically assesses the environmental effects of a product, a process or activity holistically, by analysing its entire life cycle. This includes identifying and quantifying energy and materials used and waste released to the environment, assessing their environmental impact and evaluating opportunities for improvement.

The entire life cycle of a product generally consists of three phases: production, usage and disposal. These phases can be subdivided further depending on the product and its function. The life cycle of construction products, for example, ranges from raw materials acquisition through manufacturing, construction, use, maintenance, to demolition and treatment of waste, see Figure 1.

The benefits of LCA are summarised as follows (AS/NZS ISO 14040: 1998):

- identifying opportunities to improve the environmental aspects of products at various points in their cycle;
- decision-making in industry, governmental or non-governmental organisations (eg strategy planning, priority setting, product or process design or redesign);
- selection of relevant indicators or environmental performance, including measurement techniques; and
- marketing (eg an environmental claim, ecolabelling scheme or environmental product declaration).

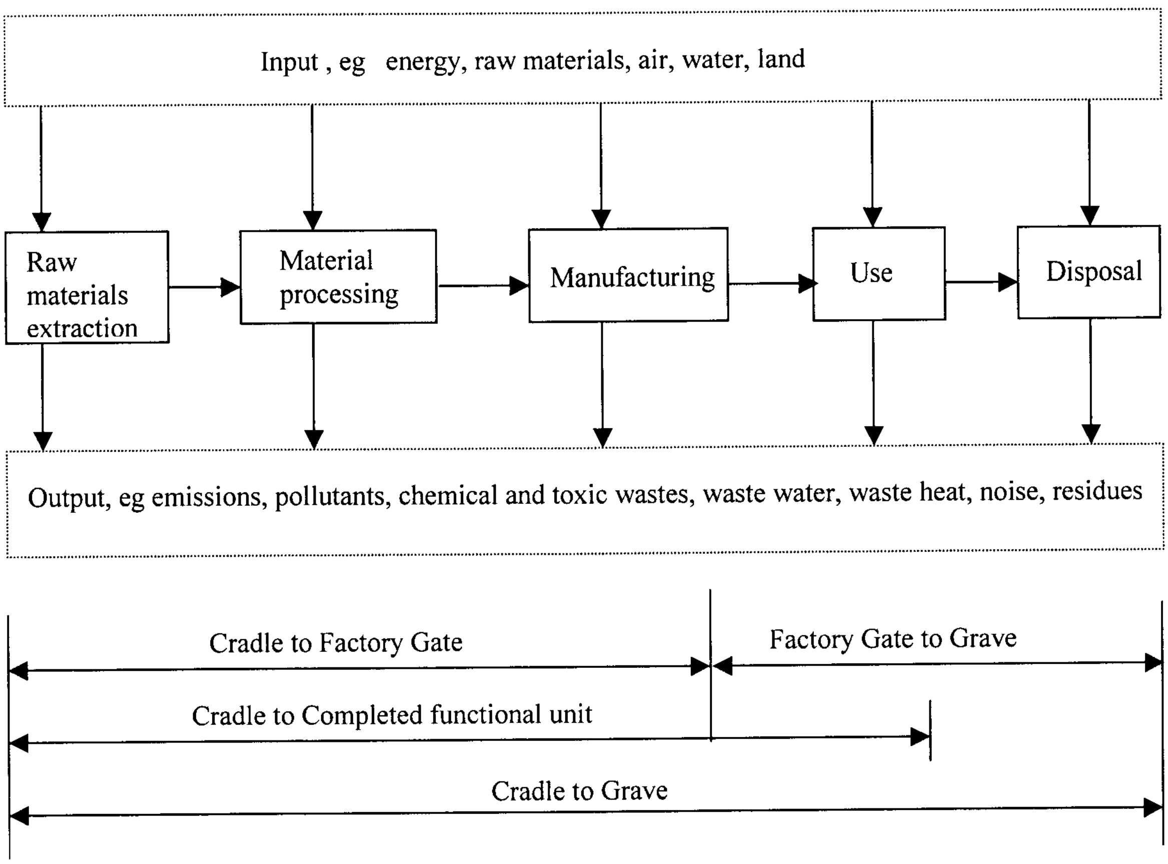


**Figure 1. Scope of life cycle assessment**

### 2.1 LCA methodology

LCA methodology comprises a number of stages or phases which are briefly described below and shown in Figure 2 (AS/NZ/ISO 14040: 1998).

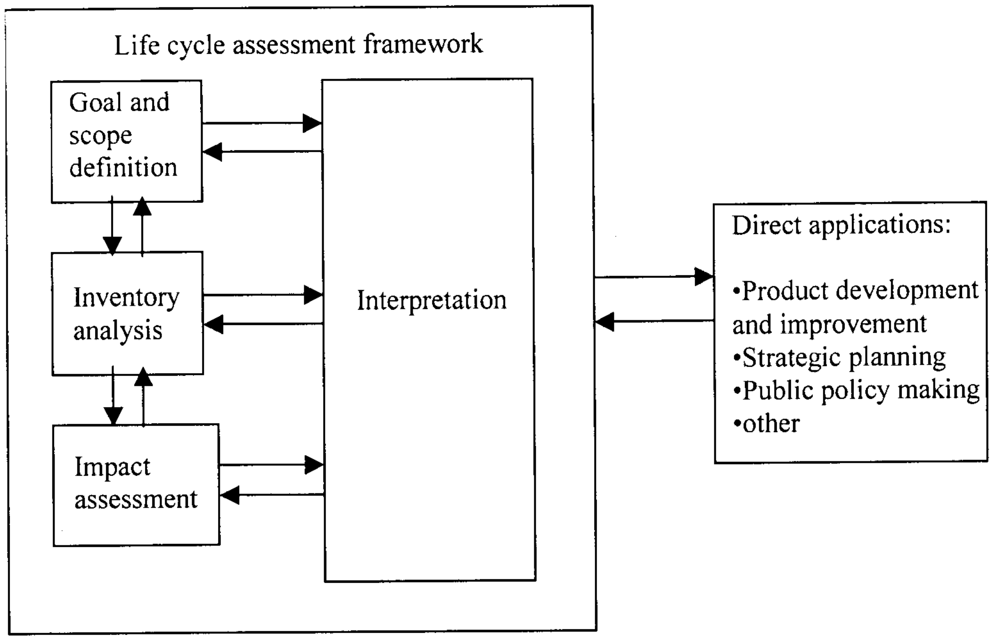


**Figure 2. Phases of an LCA (AS/NZ/ISO 14040: 1998)**

2.1.1 Initiation/Definition of goal and scope

This is a critical phase since it is at this time that the purpose of the assessment and the scope of the study are established. The boundary conditions for the study are established in this phase. The boundary conditions define the type and amount of information that will be collected in the following phases.

2.1.2 Inventory analysis

A process of quantifying materials and energy, which enter (input) or leave (output) a product system over its life cycle. Inventory analysis involves compilation of quantitative data on energy, water, all materials requirements, air emissions, waterborne effluent, solid waste and other environmental releases that occur during the life cycle of a product.

2.1.3 Impact assessment

A technical, quantitative and qualitative process to characterise and assess the effects of the environmental burdens identified in the inventory analysis phase. One of the early efforts to classify the environmental effects has been the Dutch LCA methodology (Cembureau 1995), which suggested 14 environmental parameters to be investigated. Ecoindicators 95 have been suggested by the European based organisation SETAC (The Society of Environmental Technology and Chemistry). A list of these parameters and indicators is given in the Appendix.

These parameters or measures address resource depletion, ecosystem health, human health and social health considerations. It is important to note that it is not possible to add together the effect of these parameters to achieve an overall score. Each parameter is assessed separately thus producing an environmental profile of a product. Also, the goal, scope and boundaries of an LCA study should determine the relevant environmental parameters to be investigated and need to be considered in the context of the location of the impact.

2.1.4 Interpretation/Improvement assessment

The findings of the inventory analysis or the impact assessment or both are combined to reach conclusions and recommendations within the goal and scope of the LCA study. The recommendations may include both qualitative and quantitative aspects of potential improvements such as a change in product design, raw materials usage optimisation, consumer use guidelines, waste management practices etc.

Based on these phases a typical life cycle assessment model is shown in Figure 3. As can be seen LCA is an iterative process, which could effect progressive and effective improvement in a product or a system.

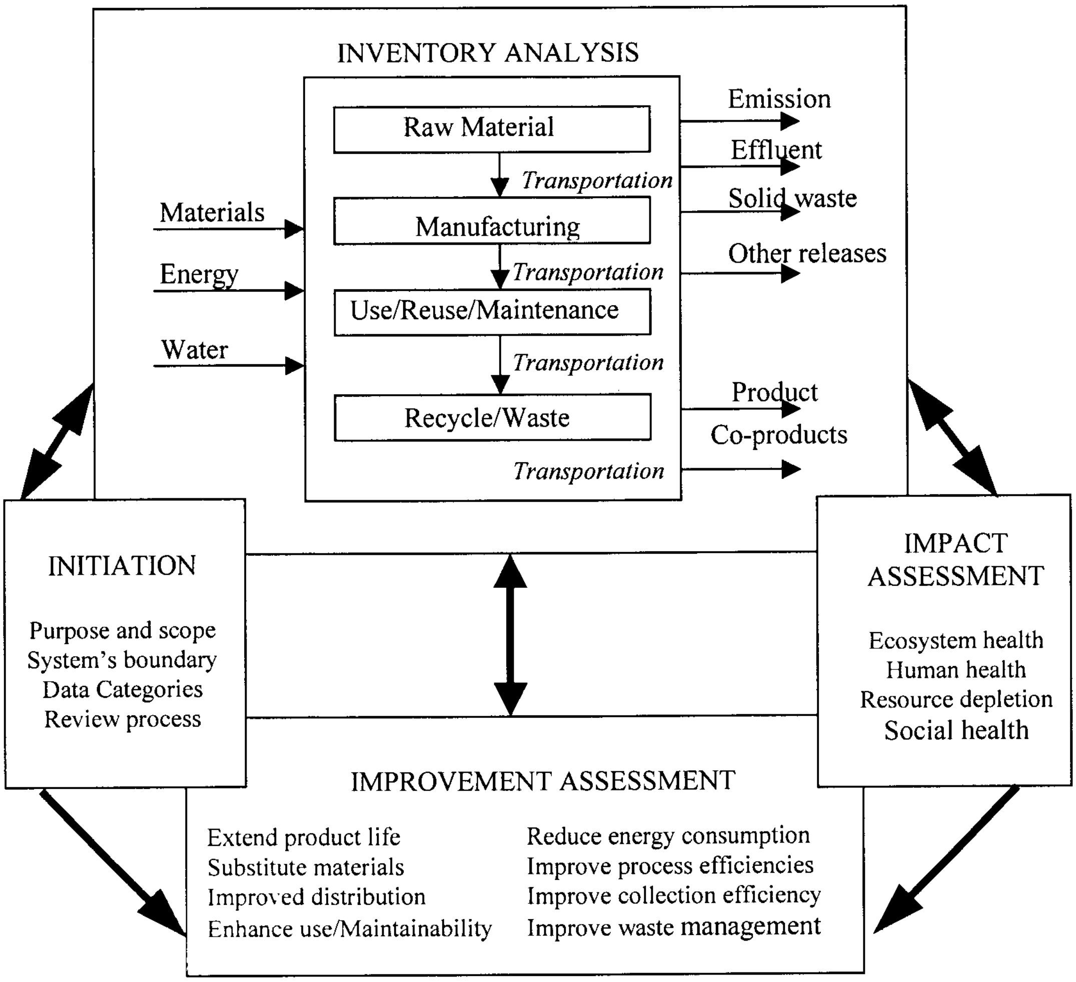


**Figure 3. A typical life cycle assessment model (Z760-1994 Life Cycle Assessment)**

### 2.2 LCA and embodied energy

In the early 90's some studies brought attention to the energy embodied within the fabric of the building. One definition of embodied energy is the total energy required by the processes of producing a product, from initial extraction of raw materials to final delivery (Lawson 1998).

The interest in embodied energy of materials and products has increased in recent years as a result of concerns for the environment. Of the various parameters, which may be used to assess the effects of materials and products on the environment, energy consumption and carbon dioxide emissions are probably the most easily estimated. Carbon dioxide emissions arising from the production of materials could be made from knowledge of the embodied energy. It is true that $CO_2$ emission is an urgent global problem and must be mitigated, but it cannot be used in isolation from other environmental impacts. Because of the ease of evaluation, $CO_2$ is sometimes chosen by itself as the criteria in the environmental assessment of two products or systems. This could lead, in some cases, to incorrect conclusions.

Within the scope of LCA shown in Figure 1, embodied energy covers a limited and in some instances an unclearly defined path. The energy embodied in a material or a product is only part of what may be attributed to buildings over their lifetimes. Life Cycle Assessment provides a more comprehensive means of analysing the energy requirements and environmental impact of buildings.

### 2.3 Life cycle assessment models and tools

The concept of LCA has only emerged on a wide scale since the late 1980's. A number of government institutions, universities, consulting firms and industries have developed LCA systems in an attempt to create an accepted standard or to demonstrate the particular benefits of their products or a combination of these.

Internationally, there is a considerable effort focused on LCA methodology with various countries and organisations developing tools and models based on the principles and framework of LCA set out by SETAC and/or ISO 14040 series. With some exceptions, most of the LCA tools developed cover mainly inventory analysis of part or all of the life cycle, embodied energy and $CO_2$ emission and/or calculation of operating energy of a building.

In Australia, the Environmental Services Group of the NSW Department of Public Works and Services, developed an LCA software LCAid™. LCAid™ is a computer modelling for environmental assessment which takes the guesswork out of sustainable building design. It allows ideas and options to be scientifically tested, while they are in the conceptual or design stages. It has the advantage of being integrated with other 3D computer aided design software such as Ecotect and Cad. Thus LCAid™ can be used to optimise design solutions.

CSIRO has developed a 3D-CAD based prototype to calculate, direct from 3D-CAD drawings, embodied energy, $CO_2$ emissions and mass for all materials in a house (Tucker and Ambrose 1998). CSIRO has also developed a software program for calculating operating energy in single dwellings (CHEENATH) and in commercial buildings (BUNYIP). To make use of these software programs in the context of life cycle assessment some major work is required, firstly to cover the whole of the life cycle, and secondly to provide the necessary "seamless" integration between the existing software to achieve userfriendliness. The Cement & Concrete Association of Australia (C&CAA) has provided funds to support CSIRO in integrating existing modules into a life cycle energy assessment tool that is also capable of being extended to provide the broader environmental assessment in the future.

## 3 LIFE CYCLE ASSESSMENT OF BUILDINGS – CASE STUDIES

The C&CAA commissioned NSW DPW&S to develop a series of case studies utilising LCA methodology and using LCAid™ as a tool.

The objectives of this study were:-

- To introduce and raise awareness in the construction industry of the "whole of life cycle" concept as applied to environmental impacts of buildings and their materials.
- To demonstrate LCA as a tool for assessing and comparing building materials and products and entire buildings over their life cycle.

The study included the following types of buildings:-

- A detached house (a typical 3-4 bedroom house)
- An office building (23 storeys, typical floor plan ~1500m$^2$)
- A warehouse (one storey, building area 12,558m$^2$)

For each type of building a number of forms of construction were considered as shown in Tables 1-3.

For each case study, assessment was made for three life cycles; 50, 75 and 100 years. A number of assumptions were made regarding building occupancy, on-site construction practices, recycling, lighting and other electricity usage. These assumptions are documented for each case study.

**Table 1. Detached house – construction options**

| | Floor | External Walls | Internal Walls | *Roof* |
|---|---|---|---|---|
| **Case 1:** | Timber floorboards | Timber/stud/plaster-board | Plasterboard | Prepainted steel roof |
| **Case 2:** | Concrete slab on ground | Brick veneer | Plasterboard | Terracotta tiles |
| **Case 3:** | Concrete slab on ground | Double brick | Rendered concrete bricks | Concrete tiles |
| **Case 4:** | Concrete slab on ground | Tilt-up panel with plasterboard and battens | Tilt-up panel | Concrete tiles |
| **Case 5:** | Concrete slab on ground | Tilt-up panel | Tilt-up panel | Concrete tiles |

**Table 2. Office building – construction options**

| | Floor | External Walls | Core and Internal Walls | *Roof* |
|---|---|---|---|---|
| **Case 1:** | Reinforced concrete slab self supporting with suspended plasterboard tile ceiling | Pre-cast concrete wall panels | Concrete core, concrete columns, infill plasterboard walls fixed to steel frames and glass partitions | Corrugated steel pre-finished roof sheet with steel structure |
| **Case 1a:** | Reinforced concrete slab self supporting with suspended plasterboard tile ceiling | Pre-cast concrete wall panels | Concrete core, concrete columns, infill plasterboard walls fixed to steel frames and glass partitions | Synthetic rubber roof membrane on concrete slab |
| **Case 2:** | Reinforced concrete slab self supporting with suspended plasterboard tile ceiling | Glass curtain wall | Concrete core, steel columns, infill plasterboard walls fixed to steel frames and glass partitions | Corrugated steel pre-finished roof sheet with steel structure |
| **Case 3:** | Reinforced concrete slab self supporting with suspended plasterboard tile ceiling | Aluminium (sandwich panel) curtain wall | Concrete core, steel columns, infill plasterboard walls fixed to steel frames and glass partitions | Corrugated steel pre-finished roof sheet with steel structure |

**Table 3. Warehouse – construction options**

| | Floor | External Walls | Internal Walls | *Roof* |
|---|---|---|---|---|
| **Case 1:** | Concrete slab on ground | Metal cladding on steel frame | Plasterboard | Metal sheet roof |
| **Case 2:** | Concrete slab on ground | Tilt-up panel | Tilt-up panel | Metal sheet roof |

Life Cycle Inventory (LCI) data used for the case studies has been sourced from the LCI data collected by DPW&S over the last five years. The data has been attained from information provided by manufacturers, describing manufacturing processes and raw material inputs. Site visits were carried out to see the processes used to assist in LCI development.

The LCI for cement was supplied by the C&CAA. It was developed in accordance with ISO standards (AS/NZS/ISO 14041: 1999).

The study has clearly shown that:

- For each building type, there was no significant difference between the different forms of construction studied in terms of energy and greenhouse gas emissions over the three life cycles considered. This was due to the fact that operation was the most important phase of the life cycle for energy usage as can be noted from Figure 4 for the detached house. Similar trends were also evident in the case studies for commercial/industrial buildings.
- Significant differences were observed in the performance of different forms of construction in terms of other environmental categories, such as ozone depletion and heavy metals.
- Environmental assessment of a building should not be based on just one or two indicators (eg energy and greenhouse gas).
- Life Cycle Assessment provides a broader view and offers a more balanced appraisal of the environmental performance of buildings.

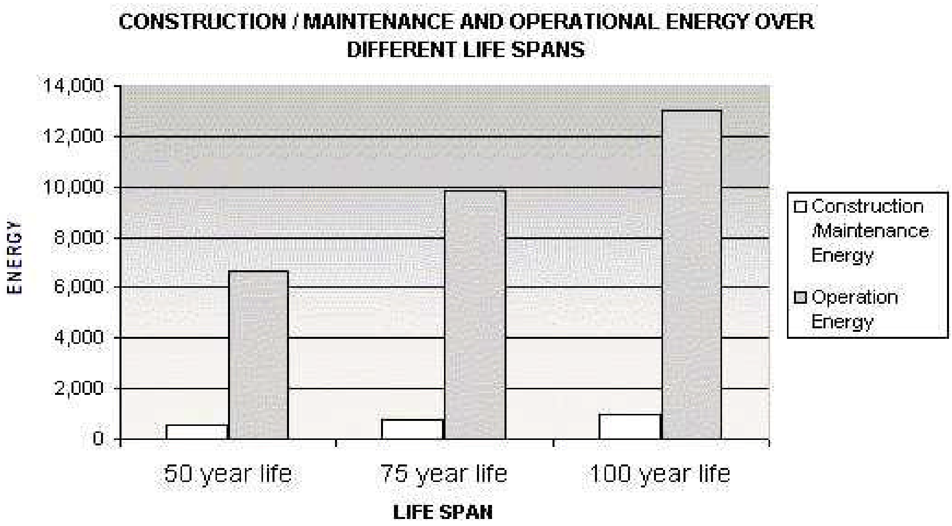


**Figure 4. Construction and operation energy for the detached house studied**

## 4 CONCLUSION AND RECOMMENDATIONS

True and relevant environmental assessment of a product necessitates the evaluation of the product in its function (ie functional unit). The functional unit defines amongst other things the size, service life, maintenance and possibilities of reuse.

Life Cycle Assessment (LCA) is developing as an acceptable methodology for assessing the environmental impact of a building throughout its life cycle encompassing extraction and processing of raw materials, manufacturing, transportation and distribution, use/reuse/maintenance, recycling and final disposal. An LCA assessment involves:

- defining and describing the product or process establishing fully the context in which the assessment is being made;
- identifying the life cycle stages being covered evaluating and quantifying the energy, water and materials usage and the environmental releases at each stage:
- determining the impacts of the releases and developing opportunities to effect environmental improvements.

The outcome of an LCA study of different building types with various forms of construction has shown that the use of one parameter or indicator is inadequate to describe or assess the environmental performance of a building (a functional unit).

On the other hand, environmental assessment as developed in an LCA study should be used as part of a more comprehensive decision process or used to understand the broad or general trade-offs. Comparing results of different LCA studies is only possible if the assumptions and context of each study are the same and these assumptions should be explicitly stated.

In an era of heightened sensitivity to the environmental impact of building products and the manufacturing processes behind them, the building industry is faced with the challenge of reaching beyond the assessment of specific building materials/products to assessment of the building itself as a functional unit.

However, the building and the construction industry at large needs to be supported in the adoption of LCA through:

- Pooling of resources (Government and Stakeholders) towards further development of LCA methodology in Australia. In particular the area of environmental impact categories relevant to Australia.
- Availability of tools and models based on the principles of ISO 14040 standards which are user friendly and capable of being fully integrated in the design process.
- Better appreciation by all involved that energy efficiency is one component of sustainable development.

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# 6 APPENDIX

In the classification process a number of standard parameters are investigated (Cembureau 1995):

(1) ABIOTIC DEPLETION: the depletion of abiotic raw materials compared with the total available resource. This is obviously relevant to the manufacture of concrete products.

(2) BIOTIC DEPLETION: the depletion of biotic raw materials compared with the total reserves and the reserves/production ratio.

(3) GREENHOUSE EFFECT: a measurement of the substances contributing to Global Warming Potential (GWP), measured as equivalent Carbon Dioxide weight.

(4) OZONE DEPLETION: a measurement of the substances contributing to Ozone Depletion Potential (ODP), measured as equivalent CFC-11 weight.

(5) HUMAN TOXICITY: an assessment of acceptable daily intakes of toxic substances by humans, measured as the equivalent body weight exposed to the toxicologically acceptable limit.

(6) AQUATIC OR TERRESTRIAL ECOTOXICITY: an assessment of substance with an eco-toxic effect on species in the ecosystem. Measured as volume of polluted water, or weight of polluted soil.

(7) OXIDANT FORMATION: Photochemical ozone creation potential of substances measured as equivalents weight of ethylene.

(8) ACIDIFICATION: Acidification potentials of substances are measured as SO2 equivalents by weight.

(9) NUTRIFICATION: Nutrification potentials of substances are measured as Phosphate equivalents by weight.

(10) AQUATIC HEAT: Waste heat emissions into water are measured as energy.

(11) MALODOROUS AIR: Odour thresholds are measured in terms of volume of polluted air

(12) NOISE: Noise is measured as an effect over time.

(13) SPACE: Use of space is measured as area over time.

(14) VICTIMS: The measurement of the numbers of human fatalities directly attributable.

## 6.1 ECOINDICATOR 95

Ecoindicator 95 was produced for the National Reuse of Waste Research Programme (NOH) in the Netherlands and includes the following impact categories.

- Greenhouse effect (Atmospheric pollution)
- Ozone Depletion (Atmospheric pollution)
- Heavy Metals (Water & Air pollution)
- Nutrification (Water & Land pollution)
- Acidification (Water & Land pollution)
- Carcinogenesis (Human health)
- Summer smog (Air pollution)
- Winter smog (Air pollution)

Additional indicators added to LCAid™ include energy and water consumption and solid wastes. This is to compare human consumption with the corresponding atmospheric and pollutant impacts.

# Implementing Material Durability Decision Support Systems For Buildings: Technical Requirements And Constraints

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**Summary: Despite the significant results from research into material durability building designers still make inappropriate material and component choices or simple mistakes. This is often due to the difficulty in accessing the wide range of knowledge required to make appropriate material selection decisions. An additional complication is the increasing knowledge in this area and the difficulty designers face in keeping up with these changes. Decision support systems have been proposed as a solution to these problems. However for these to be widely applicable material selection decision support systems will need access to data representing the building, appropriate macro- and micro-climatic data and durability models. Access will also need to be provided to the underlying choices to allow competent users to varying the underlying assumptions.**

**Keywords. Building material durability, decision support system, information technology**

## 1 INTRODUCTION

Most decisions regarding the selection and use of building materials are made by building designers, drafters and CAD operators. The level of expertise in understanding material durability issues across these groups is not high.

The idea situation would be to have automated systems that could build lists of the materials used in a building, identify the connected elements, identify the micro-climate in which individual materials and connections existed and then flag potential problems to the building designers. Automation of most of this process is currently not possible, but we will examine the various stages below. This will allow those stages which are currently feasible for automation to be distinguished from those that are not. A number of areas where more fundamental research is required are also identified.

## 2 MAKING A DURABILITY DECISION

We will use the selection of a roof cladding material as an example of the information required to make a decision that includes the durability of the cladding. We will not provide the exact data and formulae – the intention is to illustrate and discuss the process.

The product that will be assessed is BHP Billiton's Trimdek HiTen Steel cladding (BHP, 2000a). It should be noted that several of these analyses would normally be performed to assess competing products.

According to the manufacturer's literature, Trimdek HiTen is available using colorbond or zincalume finishes. We will assess the zincalume finish, which is an alloy of aluminium and zinc. The zincalume finish is given as AZ150 which according to AS1397 (Standards Australia. 2001) means that there is a minimum coating of 150gm/m2 of zincalume alloy. It is fortunate that in this instance the product manufacturer makes the required information readily available. Not all manufacturers are this considerate.

Now there are several problems that need to be resolved. Which are the critical degradation mechanisms for this product used as a roof cladding in a particular location? For these degradation mechanisms are there applicable models for estimating the durability of the component? For the models that we have, what are the critical environmental factors? Do we have data to allow analysis against these factors. When has a component "failed" against the durability factor?

To continue the example, we will examine the corrosion of the zincalume roof cladding. There is no comprehensive model for the corrosion of zincalume under Australian conditions. There is however a model for copper steel with a correlation factor for Melbourne conditions between the corrosion of copper steel and zincalume. Thus the rate of corrosion of the protective zincalume finish can be estimated. Using the data as presented in Trinidad & Cole (2002) the corrosion value for zinclaume for

Melborune is given as 0.7 µm/yr. The remaining unknown criterion is the thickness of the zincalume coating at "failure". In previous studies (Trinidad & Cole, 2002) the assumption has been that "failure" of the coating occurs when either less than 30% of the original mass of the coating remains or less than 50 gm/m$^2$ of coating remains, which ever occurs first.

From the above, the following information is required to assess the durability of materials and to select the most appropriate:

- Environmental data for the building's location
- Specifications for the product being assessed, preferably from the manufacturer
- Durability models for the degredation processes that are relevant to the material(s) in that location
- "Failure" criteria for the various relevant degradation processes

Having accessed the performance of the individual material, we also have to check that there are no incompatibilities with adjacent materials.

The manufacturer's literature for TrimDek recommends which fixings to use for zincalume sheets. When under sheet condensation is likely to occur then the sheets must be protected from contact with bare steel. Coated purlins are recommended in such circumstances. Copper and lead flashings and rain water goods are incompatible with zincalume finish and must not discharge onto zincalume surfaces.

Consequently, to assess the interaction between materials we need to know:

- Which materials are abutting (ie a model of the building),
- The micro-climate where materials abutt
- Which materials are incompatible under the particular micro-climate

Since the interaction between inappropriate materials is often complex and rapid it is unlikely that models of their interaction are particularly useful except in specific circumstances (ie using sacrificial anodes). In these circumstances heristics (rules of thumb) are often adequate, such as BHP's warning "LEAD OR COPPER ARE NOT COMPATIBLE with COLORBOND or ZINCALUME steel.".

Each of these requirements will now be discussed in more detail.

## 3 BUILDING MODEL

The most comprehensive building models currently available are being developed by the International Alliance for Interoperability (IAI, 2001). These are defined in the Industry Foundation Classes (IFCs). However, these models do not provide the detailed information required to fully automate material durability assessment. Figure 1 shows a section through a house with the standard IFC objects – roof panels, walls, floor panel and footings.

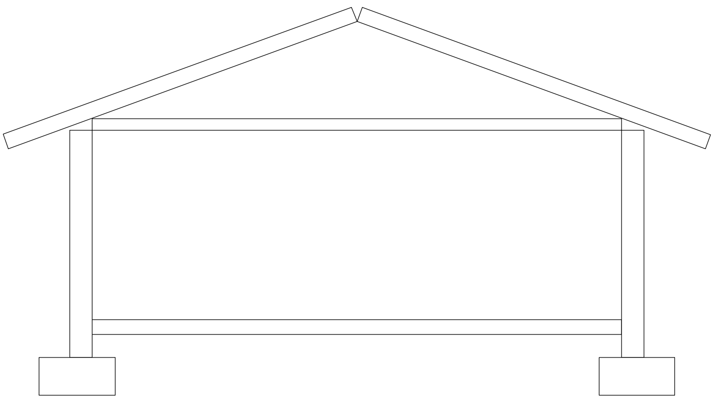

**Figure 1: Section through House with IFC Objects**

The connections between the elements are described by separate objects which can carry textual descriptions of the connection mechanism. This is not adequate for automatic assessment. For example, for a tiled roof the use of ridge and hip tiles needs to be available together with the specification of the mortar used to bed the ridge and hip tiles in place. Under current documentation conventions, the description of the mortar will be contained in a separate document, the specification, rather than the building model. This situtation is likyl to change as building models become more sophisticated and contain more information.

The layers of materials within the walls, floor and roof panels are described in a separate object that is attached to each element. This gives the material that each layer is composed of and its thickness. This information is adequate for automatic assessment if some inferences can be drawn from standard practice. For example, if the external walls are of cavity brick with a 70 mm cavity then it can be inferred that there will be brick ties spanning the cavity. If the standard spacing of air bricks is assumed then some estimate of the micro-climate inside of the cavity can also be inferred given the relevant data about the macro-environment.

## 4 THE CONCEPT OF FAILURE

A material or component has reached the limits of its durability when it has "failed". However, the concept of "failure" is difficult to define in the rigorous manner necessary in computer-based systems.

One aspect of this problem is in selecting which type(s) of failure is critical. In some uses a roofing material may be considered to have failed when the rain comes inside the structure. In other circumstances the roofing material may be considered as failing when rust starts showing on the upper surface. This will depend on the various failure mechanisms to which materials and components are subject. If we continue with roofing materials as an example, metal roofing materials will corrode at various rates and the results of the corrosion may be considered undesirable. With zinc-coated steel sheets, the appearance of rust is a good indication that the roofing needs replacement from both a corrosion and aesthetic point of view. However, with a copper roof the appearance of verdigris (the green copper oxide) is a normal and expected part of the appearance of the roof. Depending on the building, verdigris may be considered to add to the aesthetics of the building, making appearance as a durability criteria for copper roofs a very subjective assessment.

With most metals the rate of corrosion can be estimated and an appropriate value used when the component is no longer serviceable. Cementitious compounds (ie asbestos cement sheet, concrete roof tiles) and ceramics (ie clay roof tiles) do not normally fail due to loss of thickness. Due to their brittleness failure is normally caused by mechanical damage, either from stresses induced by heating/cooling cycles, by people walking on the material or by tree branches falling on a roof. The time of failure caused by these mechanisms is much more difficult to predict than for metals. A stochastic approach may prove satisfactory but this would depend on a suitable large sample size to provide reliable values.

The consideration of concrete roof tiles highlights another problem – material selection within an assembly (or "system") of components. When a tiled roof is installed the tiles are laid flat across the battens and fixed by an appropriate method. Where two planes of the roof join, such as the ridge or at a hip, specially shaped ridge tiles are bedded in mortar to cover the gap between the two planes of the roof. In this case, the weather tightness of the ridge will depend on the durability of the tiles, the mortar in which the ridge tiles are bedded and the adhesion between the mortar bed and the smooth face of the tile below. While the performance of the individual materials can be modelled relatively easily the performance of the joint is much more difficult as the long term performance will depend on not only the materials involved but the stresses imposed by differential expansion and contraction of the components.

## 5 DURABILITY MODELS

The durability of most materials used in buildings is a function of both the nature and characteristics of these products and the severity of the atmospheric environment eboth at the macro and micro scales. Optimum materials selection is determined by the required lifetime of the component or element of structure, which is usually shorter than the expected service life of the structure as a whole, and knowledge of all of the climatic and pollution factors which prevail in the environment and ultimately determine atmospheric corrosion rates (corrosivity). For the purposes of this discussion models can be divided into two categories, analytical and heuristic.

Analytical models are based on a mathematical algorithm which allows a result to be calculated given the requisite input data. For example, if the the time of wetness (TOW), sulphur dioxide pollution (SO2), and level of deposition of chloride from airborne salinity are known then the rate of corrosion of carbon steel, zinc, copper, and aluminium can be estimated (ISO 9223).

Heuristic models are built up from experience. For example the rule "carbon steel corrodes at 10.4 μm/y at locations in Melbourne more than one kilometre from the sea" gives a location specific rule. Obviously, this is not of much use ain any other location. Heuristics can be used to identify locations with similar characteristics to a location for which data exists, thus enabling the extension of an analytical model to areas for which detailed data is not available. Algorithms also exist (ie Baynes rule) for allocating a degree of confidence to such inferences. The end result of such inderences is similar to the use of statistical techniques as is used in durability maps. The advantage of heuristics is that the assumptions made in arriving at the result can be exposed to the user.

The development of models normally requires simplifying assumptions which trade off accuracy with coverage. These often need addressing as the use of models identifies shortcomings. Since the various standards define models this can be illustrated with a few examples. The present Australian and New Zealand standard dealing with the protection of iron and steel against exterior atmospheric corrosion (Standards Australia/Standards New Zealand 1994) designates three ranges of durability to indicate the duration of protection afforded by a coating to the first major maintenance viz. short term (2-5 years), medium term (5-10 years), and long term (10-20 years). This standard has however been in the process of revision for several years and is at the pre-postal ballot stage (at December 2001). The revision will most probably express durability in terms of five ranges: short term (2-5 years), medium term (5-10 years), long term (10-15 years), very long term (15-25 years), and extra long term 25+ years. A key point is made that any components of a structure not accessible after assembly should have corrosion protection that will last for the entire service life of the structure.

Another example is the ISO standard for estimating corosivity of metals. An analysis of results made from a number of studies throughout the world where simultaneous exposures of steel and zinc have been made, showed that their corrosion rates cannot always be reliably related as suggested by ISO 9223 (King G.A., and Duncan J.R. 1998). This would appear to be the case specifically in tropical and temperate marine environments, which seems related to inadequate estimation of time of wetness.

Also the balance in the existing categories presents problems. For category C2, the range of corrosivity for steel is from 1.3 to 25 mm/y and for zinc from 0.1 to 0.7 mm/y, factors of 19 and 7 respectively. The rates for steel at the bottom of the range can be found in deserts or some Antarctic sites (away from the coast), and at the top are typical of temperate climates. A range of corrosion protection systems for steel would be required in such diverse environments. The extremely corrosive environment that can prevail adjacent or very close to marine surf is not covered by the highest ISO category, and yet, often there is infrastructure built in these locations.

A limitation of the standards is that while different locations are categorized for severity with respect to uncoated steel the standards have to deal with a wide range of materials/finishes such as metal spray and hot-dipped galvanized coatings, paint coating systems, and precoated sheet metal products with metallic and metallic/organic coatings. The durability of such a diverse range of materials/finishes cannot be automatically linked to the corrosion rate of uncoated carbon steel. Notwithstanding this, guidance is provided in the standards for durability in terms of life to first maintenance for the wide range of corrosion protection systems in the various atmospheric corrosivity categories.

The ISO categories also do not take adequate account of the severity of hot dry climates with high levels of ultraviolet (UV) radiation, and tropical environments with high levels of humidity and rainfall, on the degradation of organic coatings. A special tropical category is defined in the Australian/New Zealand standards where corrosion of bare steel may be low but special consideration must be given to the durability of organic coatings.

Finally, there are still areas of material performance where the current levels of knowledge do not present the full picture. For example, the influence of initial prevailing climatic/environmental conditions on the longer term corrosion of zinc was documented 50 years ago (Ellis 1949). It was found that the ongoing zinc corrosion rate was directly affected by the “severity of the initial attack”, and this continued for at least one year. Recent CSIRO research at Port Fairy (King et al 2001, b) has dramatically confirmed this albeit at an extreme marine site. After one year the corrosion rate of zinc initially exposed in the summer (December 1999) was 37.6 µm/y, and for an exposure in the following winter (August 2000) the rate after 9 months was 8.3 µm/y. Any prediction of long term corrosion rates of zinc or zinc based coatings must take account of this phenomenon. And it is not an understatement to say that results such as these may have profound implications for the process of building in general, in marine environments, where product durability may be significantly influence by the season of installation.

In summary, developers of corrosion assessment systems will need to review their implementations as the various models are refined. There is still interesting and influential work to be done that decision support systems could assist in spreading to design practice.

## 6 ENVIRONMENTAL FACTORS

Macro-climatic data is available for large regions in developed countries but is not nearly as pervasive or accurate in less developed countries. Significant local geographical features, such as mountains, can also reduce the accuracy of climatic data. Some of the other data required, such as time of wetness, levels of pollutants, and level of salt deposition are not as widely available or reliable.

Significant problems occur when estimating the micro-climate around and within buildings. Firstly, the models for estimating micro-climates within ceiling, wall and floor cavities are not 100% reliable and secondly, the behaviour of the occupants can significantly affect the outcomes. Common problems include condensation in wall cavities due to overheating of the interiors during cold weather and the blocking of ventilation to wall cavities and under timber floors.

## 7 COMPREHENSIVENESS

A decision support system for durability that only covers a limited range of materials is not really of use to the average designer. However, more sophisticated models exist for some materials than others. For example powerful models exist for metals, while less is known about the behaviour of cementitous materials.

Rather than providing users with an “unknown” value for materials and assemblies where durability assessment is less well developed it is possible to provide heuristics (rules of thumb) that can be used to provide some guidance to less sophisticated users. This also identifies a need to indicate to users how reliable a value may be so that they are not placed in the situation where important decisions are made using unreliable information without the user being aware of it.

## 8 MATERIAL SPECIFICATIONS

Some product and material manufacturers provide all of the information and advice that is necessary to assess and use their products appropriately. Other manufacturers are not so cooperative. Some of these manufacturers can not provide reliable data due to the performance of their product depending heavily on appropriate installation or care in use. For example, floor coverings may wear very quickly if the design of a building forces large numbers of people to walk over a particular area. Other manufacturers may not be willing to provide information which shows their product at a disadvantage to their competitors.

## 9 COMMERCIAL ISSUES

There is little motivation in developing sophisticated decision support systems unless they are going to be used by a wide range of users. This brings a number of less technical but never the less important issues to the fore.

Dealing with the issue of negligence is difficult. Under Australian law, professional negligence applies when the practitioner has not applied the appropriate levels of care to their area of expertise. An adequate defence is that the decision was made at the current level expertise and knowledge that exists within the profession. This protects designers when a product is being widely used but in the longer term does not meet the claims of the manufacturers.

However, negligence issues make things more difficult if a designer claims expertise in a new area, such as durability assessment. Since there is no significant established level of expertise within the design professions, a designer claiming special expertise would be exposed to a higher risk of future claims for negligence. This would also mean that durability decision support systems would need to provide a “drill-down” capability to allow experts to understand the basis for and implications of the system’s recommendations  Of course, once durability DSS become more widely used the level of expertise required to avoid negligence claims will reduce.

## 10 CONCLUSIONS

Many aspects of materials durability assessment can be automated using existing data, durability models and software tools. However, the ability to fully automate the checking of building designs as they are represented in architectural CAD applications is still a long way off.

The areas which are currently adequately covered are models for durability of some material groups, environmental data for some countries and regions and establishment of “failure” criteria for some materials and durability factors.

Areas where more work is required include:

- detailed CAD models of buildings that allow the types, locations and adjacencies of all materials and components to be specified;
- durability models for material groups that are currently not well understood,
- collection of environmental data for regions and factors that are still missing,
- development of environmental models for the micro-climates within buildings and building components (i.e. in wall cavities, under suspended floors) and
- encouraging manufacturers to provide detailed information about their products that allows the assessment of their durability.

Material durability decision support systems that are not comprehensive will have difficulty finding acceptance with the average building designer. This means that sections of the “body of knowledge” must be built up over time until comprehensive systems can be offered to the industry.

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# The Durability of Building Wraps: An investigation of Temporary Weather Protection to Buildings Under Construction

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**Summary: Breather-type building wraps have been used in most systems for housing and low-rise construction in New Zealand for many decades. The general view is that these wraps play an important role in helping control moisture transfer through the building envelope. Key properties have traditionally been: water vapour permeability, water absorption, strength and liquid water resistance.**

**The material of choice for building wraps until recently has been bitumen-impregnated Kraft paper. These products generally perform well but do have limitations with regard to extended wetting by rain or building leaks. Strength losses of the order of 65% on wetting are typical for paper-based wraps, and loss of integrity during wind and rain is common, with subsequent loss of wrap from the building frame during construction. Synthetic alternatives based on materials such as spunbonded woven or non-woven polypropylene have become popular in the last five years in the belief that they are resistant to weather exposure and do not suffer strength losses. These materials have a high burst strength which is unaffected by water and some are available in widths which can cover an entire storey height. These properties result in a wrap that is attractive as a temporary weather protection for buildings under construction. Some builders have adopted an uncommon practice where insulation and interior linings are being installed prior to the exterior cladding. This is a concern since the weathertight performance of the wrap during the construction phase is unknown, and insulation and linings may suffer water damage and water may be trapped in the cavity once it is enclosed.**

**New Zealand Standard NZS 3604: 1999 and E2/AS1 (Acceptable Solution 1 to New Zealand Building Code Clause E2 "External moisture") have requirements for building wraps. These have limited performance criteria: that of water vapour transmission resistance [WVTR] and water absorbency. When tested in accordance with AS/NZS 4200, both bitumen paper and spunbonded non-woven polyolefin wraps fail to perform in some important properties if they have been exposed prior for two to three months. This is a concern since over 80% of wraps in the New Zealand market are bitumen paper or spunbonded nonwoven polyolefins, and many construction sites leave the building wrap exposed for up to three months. It is also a concern for Australia where wrap-loading classification is based primarily on tensile strength. Both New Zealand and Australian**

**Standards should specify more stringent performance criteria, for both new and aged [90 days-UV exposure] wraps, with ">90% of new" properties retained after exposure suggested as the minimum performance level**

**Keywords: Building wraps, durability, weathertightness, penetration**

## 1 INTRODUCTION

The various roles attributed to building wraps are a second line of defence against rain water leaks, assisting with condensation control, preventing wind-wash of insulation and temporary weather protection during construction. The required properties of building wraps to meet these roles include water absorbency and watertightness. Wraps must also act as a moisture breather, allowing water vapour to escape the cavity by diffusion, yet also be a moisture barrier to limit solar-driven moisture vapour flows into wall cavities where necessary. Consequently the key performance criteria of wraps are water absorption, burst strength, liquid water permeability, air permeability, water vapour permeability and durability. The water-resistance properties of the building wrap are measured by static head [water resistance-Cobb test], water vapour transmission resistance [WVTR] and surface water absorption. Durability, while it is not specified in any New Zealand Standard, is proposed by the author to be determined by comparing these properties before and after various exposure regimes. A Canadian Standard for breather-type membranes (National Standard of Canada. 1977) requires accelerated ageing of wraps and testing for change in WVTR after exposure. A USA manufacturer [DuPont] of synthetic wraps specifies durability in their technical literature (Lotz. 1991) as a requirement for polyolefin-based wraps to retain >90% of their performance after exposure. This paper investigates the validity of 60-day and 90-day weathering regimes, which includes a BRANZ *ad hoc* testing procedure (Bassett, 2000) using a purpose-built wind-rain rig.

The building wraps tested are examples of materials that fit into two generalised classes, in accordance with their method of manufacture and make-up. While other countries, like the UK, have also classified such products as ‘Types or class A/B’ based on performance (BS4016: 1997, BS1521-1972), the performance of bitumen-based papers can be very similar to many polyolefin synthetic wraps in some tests, but quite different in others, or when wet, depending on their make-up. The Type-classification proposed in this paper defines Type 1 material as bitumen-impregnated Kraft paper and Type 2 as spunbonded non-woven polypropylene or polyethylene. The performances of Type 1 wraps are primarily subject to changes in grammage [$g/m^2$], thickness and degree of bitumen saturation, while those in Type 2 class can vary in some properties with changes in chemical additives, pore volume variations, fibre density, thickness and grammage.

The function of a wrap to provide a degree of ‘Temporary Weather Protection’ [TWP] to the building is not a requirement of the New Zealand Building Code [NZBC] or the Standard that specifies materials for the building envelope, NZS 3604 [SNZ. 1999]. However, the market sees this as a desirable property and a test procedure has been developed by BRANZ (Bassett, 2000) to evaluate it. This test is particularly desirable since many manufacturers have commented on builders fixing the wrap and fitting internal linings up to two months before the external cladding is fixed. The TWP testing involves weathering for up to 90 days, with wind resistance, water absorption, water-penetration resistance and air permeability testing before and after exposure.

The actual samples of wraps used in this study that represents the two Types are:

| | |
|---|---|
| Sample A | Type 1 260 gsm |
| Sample B | Type 1 450 gsm |
| Sample C | Type 2 75 gsm |
| Sample D | Type 2 100 gsm |
| Sample E | Type 2 100 gsm |
| Sample F | Type 2 150 gsm |

Therefore the Type 2 [75 gsm] data refers to that obtained for Sample C. Samples D, E and F are almost identical in their manufacture, make-up and performance, therefore the test data for the 100 gsm and 150 gsm Sample D, E and F products are combined under the classification of ‘Type 2 100 gsm’, or ‘Type 2 150 gsm’, respectively.

## 2 METHODOLOGY

The TWP exposure procedure fixes the wraps to a 2.4 x 2.4 m timber frame, with 600 mm stud spacing, and placed on the BRANZ Judgeford natural exposure site, 41$^o$ 10’ S latitude and 174$^o$ 55’ E longitude, facing north, standing vertically, for three months, preferably during the winter/spring period. The wrap sample was fixed using staples and polypropylene [PP] strapping; however when staple tear occurred from wind gusts, repairs were undertaken using clout nails and washers. This exposure provides up to three months of natural UV ageing, with wind, heat and rain stresses. The frame is then removed intact from the site and placed 4 metres in front of an electric fan capable of delivering air speeds of up to 20 m/s over a 1.44 $m^2$ wrap area. In the trial, eight sampling regions on the wrap, each spaced 300 mm apart within an area of 1.44 $m^2$ at the centre of the wrap, were tested for pressure differential through the wrap, and air velocity in front of, and behind the wrap, at wind speeds of 10 m/s [36 kph] and 20 m/s [72 kph]. The rain penetration was observed at these air speeds when a water system in front of the fan outlet delivered a water spray of both fine and heavy droplets into the moving air mass. After pressure differential, wind velocities and rain penetration were recorded the wrap was removed from the test frame and the water penetration resistance retested in the laboratory.

The air velocity was measured using a Dantec flowmeter, model # 54N60, and the pressure differential was determined using a calibrated manometer system. The wrap thicknesses were measured with a calibrated digital calliper while the pore area and open area were measured under an Olympus BH optical microscope using an Olympus Measure Master software package. The

Gurley apparatus of BS 6538: Part 3 (BSI. 1987) was used to determine air permeability. The test methods used for the laboratory-based testing were AS/NZS 4201.4 (SA. 1994) [water-penetration resistance], AS/NZS 1301.448 (SA. 1991) [tensile strength], AS/NZS 2001.2.19 (SA. 1988) [ball burst strength], TAPPI T470 (TAPPI. 1989) [edge tear strength] and AS/NZS 4201.6 (SA. 1994) [surface water absorbency] in accordance with specifications defined in AS/NZS 4200 (SA. 1994).

A sample of 260 gsm and 450 gsm Type 1 wraps were fixed to frames and exposed to the weather at Judgeford. However Kraft-based papers have poor durability and edge tear resistance when wet, therefore no sample survived for a long enough time to enable meaningful weathering exposure worthy of testing. A sample of Type 2 [100 gsm] and [150 gsm wraps] were exposed for two months, on 1m x 1m frames standing vertical, facing north, in Auckland and then tested at BRANZ. Another Type 2 wrap and the 75 gsm Type 2 wrap was exposed for 90 days weathering in Wellington, then tested using the TWP procedure. The aim is to get an indication of the comparative performance of several samples of wraps of a similar make-up, between 'off-roll' and after weathering for the two- and three-month periods.

## 3 RESULTS

The 'baseline' performance parameters of building wraps supplied 'off-roll', before the weathering, are recorded in Table 1, from tests at BRANZ. The weight of the wraps is measured in $g/m^2$ units [gsm]. Wraps are also 'directional' as a consequence of the production process; therefore 'machine direction' defines the line parallel to the movement of the production line.

**Table 1 Baseline performance of building wraps tested- unweathered wraps**

| Product | Water-liquid penetration | Water vapour transmission [WVTR] range (MNs/g) | Surface water absorbency range $g/m^2$ | Air permeability $MNs/m^3$ |
|---|---|---|---|---|
| Type 1 [260 gsm] | Pass | 0.9 - 1.0 | 150 - 155 | 1.3 |
| Type 1 [450gsm] | Pass | 0.7 - 4.0 | 165 - 530 | Not tested |
| Type 2 [75 gsm] | Pass | 0.3 - 0.8 | 123 | 0.3 |
| Type 2 [100 gsm] | Fail | 0.2 - 0.3 | 100 - 160 | Not tested |
| Type 2 [150 gsm] | Fail | 0.15 - 0.2 | 100 - 155 | 0.3 - 0.4 |

### 3.1 Natural weathering of a Type 2 wrap after two months

Tables 2-3 list some performance parameters for a selected Type 2 wrap before and after two months weathering. After two months weathering the Type 2 wrap sample suffered an approximate 10 - 12% reduction in burst strength and tensile strength. The reduction in strength reflects increased brittleness of the polymer.

**Table 2 Water properties of a Type 2 wrap before and after two months weathering**

| Weight | Exposure | Resistance to water penetration | Water absorbency range ($g/m^2$) |
|---|---|---|---|
| 100 gsm | Off roll | Fail | 139 |
| 100 gsm | Weathered | Fail | 270 - 320 |
| 150 gsm | Off roll | Fail | 136 |
| 150 gsm | Weathered | Fail | Not tested |

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**Table 3 Strength of a Type 2 wrap before and after two months weathering**

| Weight | Exposure | Orientation | Mean tensile str kN/m | std dev | Burst strength to AS 2001.2.19 (N)-Wet* | Std dev | Burst strength to AS 2001.2.19 (N)-Dry* | std dev |
|---|---|---|---|---|---|---|---|---|
| 100 gsm | Off-roll | md | 3.07 | 0.12 | 249.0 | 18.0 | 241.0 | 20.0 |
| | | cd | 2.04 | 0.06 | | | | |
| 100 gsm | weathered | md | 2.67 | 0.07 | 217.8 | 20.0 | 216.1 | 13.0 |
| | | cd | 2.00 | 0.05 | | | | |
| 150 gsm | Off-roll | md | 3.50 | 0.14 | 300.1 | 25.0 | 302.3 | 17.0 |
| | | cd | 2.57 | 0.08 | | | | |
| 150 gsm | weathered | md | 3.16 | 0.14 | 278.0 | 7.0 | 274.1 | 20.0 |
| | | cd | 2.25 | 0.05 | | | | |

***** The burst strength results are averaged for machine and cross-directions.

### 3.2 Temporary weather protection testing for a Type 1 wrap

Water pentration performance of a Type 1 [260 gsm] wrap tested on the TWP rig with air speeds up to 24 m/s [86 kph] are recorded in Table 4.

The low weight Type 1 wrap began tearing at the fixings when the wind speed reached 15 m/s [137 Pa]. The air permeability of this Type 1 wrap, as measured by the Gurley apparatus in accordance with BS 6538. Part 3 (BSI. 1987) is 1.0 – 1.4 MNs/m$^3$, with an average of 1.3 MNs/m$^3$. A Type 1 [450 gsm] wrap remained intact and reasonably dry between the 600 mm stud spacing and does not suffer ‘wet burst’ up to a wind speed of 24 m/s if fixed by clout nails, washers and polypropylene straps. Note that the width of Type 1 wraps is shorter than the storey height; therefore the mandatory 75 mm minium overlap exists on the test frames.

**Table 4 Water penetrations for a Type 1 [260 gsm] wrap**

| Air speed [Peak value] m/s | Comment |
|---|---|
| 13 | Watertight - splashing occurs through overlap |
| 15 | Tearing and leaking at edges near frame - splashing more severe |
| 18 | Copious leaking at edges and overlap |
| 24 | Total failure as entire edge tears away and wrap blows off |

### 3.3 Natural weathering of selected Type 2 wraps after three months

TWP testing results are shown in Table 5 at air speeds corresponding to 0, 10 and 20 m/s. The free stream pressures indicated in Table 5 are calculated from the expression $P= 0.61v^2$, where v is the wind speed [m/s] in accordance with NZS 4203 (NZS. 1992). The TWP test rig is shown in Figure 1 and a weathered Type 2 [100 gsm] wrap is shown in Figure 2.

#### 3.3.1 Type 2 [75 gsm] wraps

Air permeability of a Type 2 [75 gsm] wrap, as determined in accordance with BS 6538. Part 3-1987, was 0.3 MNs/m$^3$ [off-roll]. US product specification (Lotz. 1991) for a Type 2 [75 gsm] wrap indicate an air permeability of approximately 63 - 65 sec/100 mL, which is approximately 0.5 MNs/m$^3$. The air permeability of the same Type 2 [75 gsm] wrap after 90-days exposure using the standard Gurley method was 0.3 - 0.4 MNs/m$^3$, suggesting that air permeability is not affected by UV degradation of the wrap. The Type 2 [75 gsm] wrap sample remained water tight up to wind speeds of 20 m/s.

#### 3.3.2 Type 2 [100 and 150 gsm] wraps

The Type 2 [100 gsm] and Type 2 [150 gsm] wrap samples tend to bead water well on the outer surface when exposed to wind-driven rain, but when wind pressure is increased, water penetration occurs, especially at points where the material makes contact with the timber stud and there is a change in surface tension. Wind-driven water is also blown between the layers where the overlap occurs, since most Type 2 [100 gsm] and [150 gsm] wraps tested were shorter in width than the storey height. This suggests that the current overlap requirements in NZS 3604 are inadequate as far as temporary weather protection is concerned, and need to be increased when using Type 2 [100 gsm] and Type 2 [150 gsm] wraps. The overall result is that most Type 2 [100 gsm] and Type 2 [150 gsm] wraps tend to fail in preventing water ingress, even at low wind pressures.

It was observed that the Type 2 [100 gsm] and Type 2 [150 gsm] wraps fixed with a 175 mm overlap at the centre were capable of surviving extremes in natural wind pressures from both front and rear sides, since most of the pressure is reduced by the overlap, but the overlap allows wind-driven water to penetrate.

**Table 5. Water penetration for a Type 1 and a Type 2 wrap [100 and 150 gsm] on the TWP rig**

| | Exposure time - days | Weight - g/m$^2$ | Free - stream pressure – Pa | Water penetration comment |
|---|---|---|---|---|
| Type 1 | 0 | 260 | 55 | tight |
| Type 2 | 0 | 75 | 55 | tight |
| Type 2 | 0 | 75 | 220 | tight |
| Type 2 | 0 | 100 | 55 | Fine spray penetration |
| Type 2 | 0 | 100 | 220 | leaks |
| Type 2 | 0 | 150 | 55 | Fine spray penetration |
| Type 2 | 0 | 150 | 220 | leaks |
| Type 2 | 90 | 75 | 55 | tight |
| Type 2 | 90 | 75 | 220 | tight |
| Type 2 | 90 | 150 | 55 | leaks |
| Type 2 | 90 | 150 | 220 | leaks |

**Figure 1 TWP test rig- Judgeford**

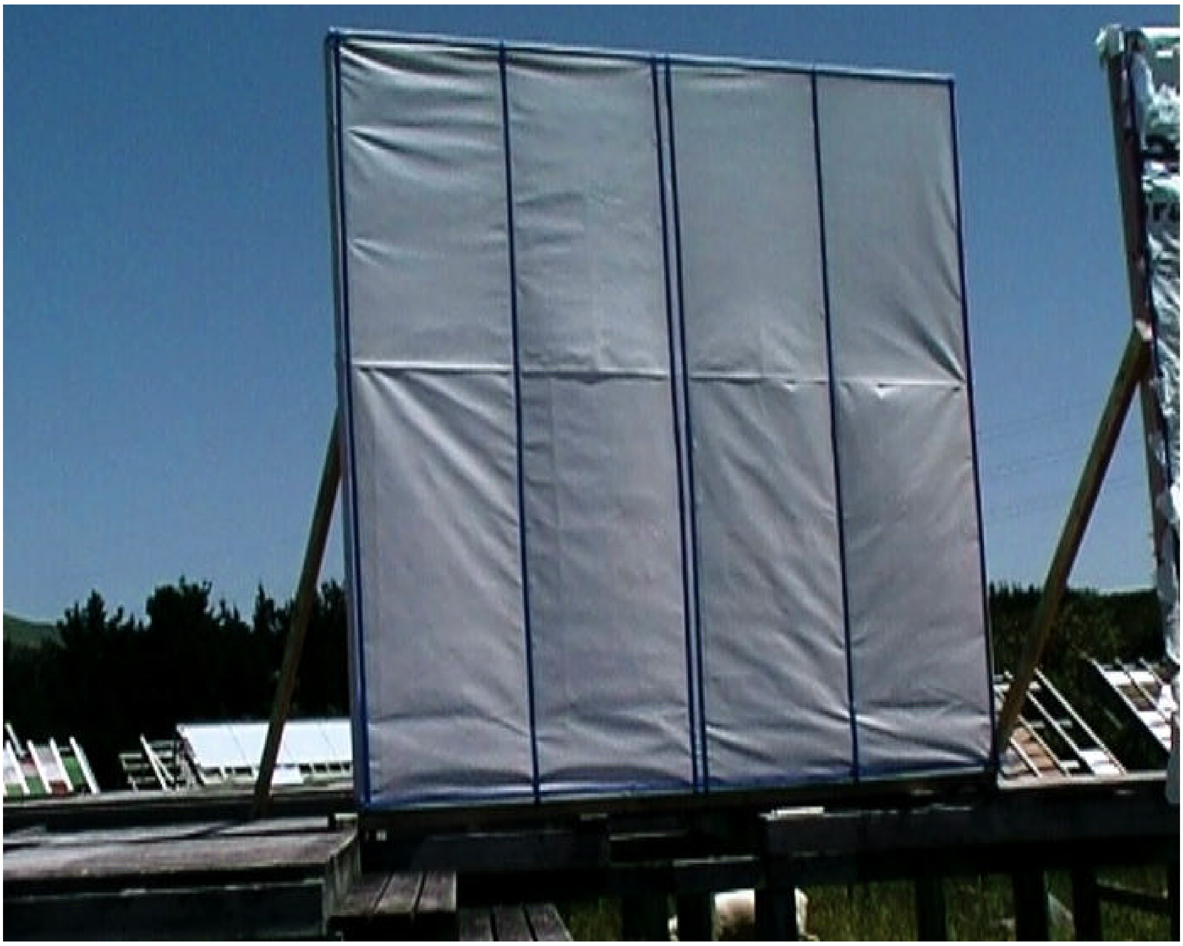

**Figure 2. A Type 2 [100 gsm] wrap weathering at Judgeford**

## 4 DISCUSSION

The consequence of natural exposure of the selected wraps during the construction phase is shown in Table 6. It is suggested from this preliminary trial that building wraps may pass most performance criteria defined in AS/NZS 4200 when supplied from the store, but they can potentially fail if left exposed to the weather for an extended period. This period could be between two to three months. The New Zealand Building Code, clause B2 requires building wraps to meet a durability requirement of at least 15 years, and in some cases 50 years. This period is normally considered to commence once the material becomes part of the building envelope. However, these preliminary tests suggest that a two to three month delay in enclosing the structure with the external cladding potentially degrades the building wrap, such that it may no longer be ‘fit for purpose’, as defined by AS/NZS 4200 (SA. 1994) and possibly fail to meet the durability period of 15 or 50 years while in service.

The TWP tests suggest that the spunbonded non-woven embossed materials, such as the Type 2 [100 gsm] and [150 gsm] wraps, or any material with significant pore sizes, leaks more severely than would be anticipated from results of the water-penetration resistance test. This water-penetration resistance method uses a fixed head of water pressure of 0.98 kPa. That is, there is no correlation between the water penetration and the TWP results. More specifically, the wind pressures of 55 Pa and 220 Pa in the TWP procedure are equivalent to a static head of water of only 5.6 mm and 22 mm, respectively, and yet the TWP procedure records a ‘fail’ for water penetration resistance at both of these low wind pressures. This is best explained by the suggestion that the wind-driven rain drops in the TWP procedure carries a momentum that applies a larger localised pressure to the wrap than the overall kinetic pressure of the wind. That is, materials could easily pass the AS/NZS 4201.4 (SA. 1994) requirement with a 100 mm head of water, but fail the TWP procedure at wind speeds between 10 m/s and 20 m/s, likewise there is no correlation between TWP and water penetration resistance or WVTR. However, there may be a simple relationship between air permeability and water vapour permeability [see Table 1], as both are influenced by pore size, and pore size may be altered slightly while under wind stress in the TWP procedure. The differences in manufacture, structure and make-up lead to differences that are not necessarily visible, yet they make it difficult to draw any correlations between these performance parameters. Further work is required to establish any relationship between these parameters and the wraps structure and make-up.

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**Table 6 Summary of compliance to Standards before and after two to three month exposure**

| | AS/NZS 4200 criteria | AS/NZS 4200 criteria |
|---|---|---|
| | **Before** | **After** |
| Type 1 [260 gsm] | Fails edge tear & wet burst [ball] | Fail |
| Type 2 [75 gsm] | Pass | Pass |
| Type 2 [100 gsm] | Some are 'borderline fail' or fail tensile md & cd and static head | Fail |
| Type 2 [150 gsm] | Some are 'borderline fail' or fail tensile md & cd and static head | Fail |

## 5 CONCLUSION

TWP testing shows that differences exist between the various types of wraps available in the New Zealand market. The low weight [75 gsm] Type 2 wrap commonly used, which is made from high-density polyethylene, meets all performance expectations. The performance variations shown in these tests are consistent with anecdotal evidence being reported from building sites in New Zealand.

The New Zealand Standards NZS 3604 (NZS. 1999) and NZBC E2/AS1 (BIA. 2000) specify limited performance criteria that are not adversely affected by weathering during the construction phase. However, when tested against NZS 4200 (A/NZS. 1994), both samples of Type 1 [260 gsm] and Type 2 [100 and 150 gsm] wraps tested suffer reduction in tensile strength when exposed for up to two to three months. This is a concern since over 80% of wraps in the New Zealand market are of Type 1 or Type 2, and many construction sites leave the building wrap exposed for up to three months. It is also a concern for Australia where wrap-loading classification is based on tensile strength [Schafer. 2000].

Both New Zealand and Australian Standards should specify more stringent performance criteria, for both new and aged wraps, with a suggested limit of '>90% of new' properties retained after 90-days exposure, as the minimum performance level. However, it should also be recommended that external claddings must be fitted before 60-days exposure. This limit should be set regardless of the deterioration mechanism, which may be different for Type 1 wraps compared to Type 2 wraps. The test data for all commercially acquired products are consistent with their technical specifications and claims.

## 6 ACKNOWLEDGMENTS

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# Rain Penetration Performance Of Mortarless Interlocking Blockwork Walls

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**Summary: This paper describes the observations on a rain penetration test on walls built from interlocking blocks. The test was conducted on four walls: three built from plain concrete interlocking units that were unplastered, plastered on internal face only, and plastered on both internal and external faces, respectively. The forth wall, unplastered, used block units from concrete with water proofing admixture. The results of tests on the unplastered and the internally plastered walls from plain concrete units showed very poor resistance to water penetration. Proper plastering on the external face of the wall provided acceptable rain penetration resistance. The wall from water proofed units provided better resistance to water penetration by absorption. Nevertheless, water penetration through the mortarless joints was still of concern.**

**Keywords. Block, blockwork, mortarless, interlocking, rain penetration.**

## 1 INTRODUCTION

The system of utilising load-bearing interlocking mortarless blockwork in construction is not uncommon in the United States and Europe. Some of the popular brands of interlocking block units found in those countries are Haener, McGibbs and Sinustat. A number of housing projects in the Philippines and Thailand were found to use the system. Basically, the system used interlocking concrete block units that were laid dry without any mortar for bedding and jointing. The interlocking features were made either at bed joints (horizontal joints) or header joints (vertical joints) or both.

The system was first introduced to Malaysia in the 80's as a prospective answer to the growing demand on low-cost housing. Initial work was to find a simple and cheap D.I.Y. (do-it-yourself) method of construction (Wardi 1987). Although the interlocking block unit costs more than the conventional block unit, the construction of the former saved time and labor which resulted in overall saving. Despite that, developers were skeptical to invest in the system and as a result only few units of such houses have

been built. The use of the system in building construction in this country seemed to never get off the ground. There were several factors that led to this, among others were:

- Lacking of knowledge or experience amongst engineers on the design of load-bearing masonry structures.
- No standard design procedures and guidelines were available.
- Structural and serviceability performances of the system were uncertain. Developers and builders were not willing to invest in R&D works on the system.
- Model units built were limited to single-storey building only and the performance of these units have not been convincing. A case of a pilot project based on Wardi's (1987) concept failed due to improper foundation design seems to have put developers and builders off.

Recently, interests in the system have been rejuvenated inspired by the American experience. Several studies (Azzam *et al* 1993, Maju 1994) on the system were carried out to ascertain the viability of using such system to suit the local climatic and social environments. One of the initiatives was by Maju Holdings (1994) whose immediate interest was to look into the possibility of utilizing the interlocking system for the construction of buildings up to 5-stories high. The company tested the interlocking block units and walls to study the structural performances, fire ratings, resistance to rain penetration, build-ability and others. The block units, called the 'Maju Blocks' were specially manufactured and shipped from the United States. These units were smaller and lighter as opposed to the US specifications in order to suit handling by local workers. Earlier tests carried out showed that the system attained the required structural capability for the construction of buildings up to five stories high (Abdullah *et al* 1995). The rain simulation test described in this paper was part of the study carried out for Maju Holdings.

## 2 FUNCTIONAL REQUIREMENTS OF INTERLOCKING BLOCK UNITS

The design of interlocking block units must achieve several qualities that include (Maju 1994):

- axial load-bearing capacity
- efficient transfer of axial loads
- interlocking aid to alignment
- waterproofing at joints
- continuity of cores for grout filling
- good keying to provide nominal lateral shear resistance
- tolerances to maintain wall alignment
- simplicity of form for manufacturing
- suitable size for handling and ease of construction.

## 3 WATER PENETRATION THROUGH MASONRY WALLS

Water penetration is responsible for many of the problems encountered in masonry construction (Ritchie 1952). Moisturized walls subjected to freezing and thawing that may cause cracking, crazing, spalling and disintegration. Water can cause masonry to experience numerous effects such as shrinkage, corrosion of steel reinforcements and other metals built in the walls, loss of effectiveness in insulation, deterioration of the interior finishes, efflorescence and fungi growth, dampness and other problems.

### 3.1 The mechanism of rain penetration (Waldum 1995)

Water at the wall surface can penetrate the wall by several means including capillary action, gravitational flow, kinetic energy as well as water flow. When rainwater hits a masonry wall surface, the water that wetted the wall is first sucked into the wall material. If the rain continues faster than the suction, the water starts to run down the wall, forming a film of water, which is thicker on the lower part of a building than the upper part. This film of water forms a bridge over the unavoidable small cracks in the masonry wall. Wind acting on the wall forms a pressure difference over the wall. By this pressure, the water is forced into the wall. Rain penetration occurs in cracks between 0.1 and 5mm wide. The wind pressure is added to capillary suction. The latter seems to be important for openings smaller than 0.5mm.

With water-filled pores, a slight pressure difference is sufficient to make the water discernible on the inside of the wall. Factors that affect rain penetration are the properties of the wall materials, the amount of water hitting the wall, and wind pressure over the wall. The intensity of driving rain on a given surface determines whether a water film will be formed and the thickness of that film. The amount of water striking the wall during a longer period determines, in certain cases, the amount of water collected in the wall. Wind pressure is dependent of the velocity of the wind, the orientation, shape, height and exposure condition of the building.

### 3.2 Rain penetration on mortarless masonry

Rain penetration takes place in two ways: firstly, water absorption through the units and secondly, water seepage through the joints. The former problem can be overcome through proper material design such as using high density concrete, use of water repelling admixtures, brushing the wall with water repellent paint or solution and plastering the external face of the wall. The later problem is the main concern with mortarless interlocking masonry. The mechanism of water penetration through the mortarless joints would be expected to be similar as for penetration through cracks found in the traditional masonry wall.

For conventional masonry, the water penetration resistance is provided by the mortar joints. As for the mortarless masonry system, the design of the interlocking features is important in terms of rain penetration as well as fulfilling its other functional requirements stated earlier. It has been difficult to design interlocking features that provide good rain penetration resistance without affecting the other requirements.

Several methods have been found suitable to overcome the problem of rain penetration in mortarless masonry. In the U.S. the ‘dry-stack’ system and the surface bonded system are used. In the former system, the grouting of all cores stipulated as a standard construction procedure provides the water penetration resistance, while the latter system requires plastering on both wall faces with a strong mortar laced with chopped fiberglass for strength to provide the necessary water-tightness (ASTM 1991). Special premixed mortar products for this purpose are readily available on the US market. For a single-storey building, a longer roof eave construction had proven successful in protecting the walls from direct splash and long hours of rain as shown by a model house (Wardi 1987).

## 4 EXPERIMENTAL DETAILS

It should be pointed out that, the block units in this study were design and manufactured with the external stretcher face 'finished' without requiring any external plastering. This was to keep the cost of construction down as external plastering would incur about 20% additional cost on the wall construction. However, when preliminary tests indicated poor water penetration resistance, cheap alternatives to provide water penetration have to be looked into.

The objective of the tests was to evaluate the rain penetration performance of the interlocking blockwall system using a simple, cheap and commonly practiced construction method. Tests were conducted with the following purposes:

- To evaluate the effectiveness of the interlocking block construction against water penetration.
- To study the effect of plastering on water penetration: (i) internal plastering only, (ii) plaster both internal and external faces.
- To evaluate the performance in terms of water penetration, using the block with water proofing admixture.

### 4.1 Block units

Two types of block units were used; the stretcher and corner units for external walls. The stretcher unit had interlocking features at top (tongue) and bottom (groove) of the bed faces and on both of its header faces. The corner unit had similar horizontal tongue and groove features as the stretcher unit but with only one of the header face having vertical interlocking features. The vertical interlocking feature (tongue and groove feature running in the vertical direction) was made on the internal side of the stretcher faces. The features of the units are shown in Figure 1. In addition, two different types of mix were used in the manufacturing of the units: (i) plain concrete and (ii) concrete with water proofing admixtures. The block properties are given in Table 1.

**Table 1. Properties of block units**

| *Block Type* / *Properties* | *Plain Concrete* | *Concrete With Water Proofing Admixture* |
|---|---|---|
| Weight of Unit (dry) | 11.8 kg. | 11.9 kg. |
| Water Absorption (24 hrs immersion) – by wt. of water. | 5.9 % | 2.5 % |

### 4.2 Test walls

Test walls were constructed at an open area exposed to the ambient temperature and humidity. The walls comprised four corner (L-shaped) walls of about 1.2m by 1.2m, 1.5m high. A normal lime mortar mix with 12mm thickness was used. The construction details of the test walls are given in Table 2.

**Table 2. Test Wall Details**

| *Designation* | *Type of Block Unit* | *Plastering* |
|---|---|---|
| Wall 1 | Plain concrete | None |
| Wall 2 | Plain concrete | Internal face |
| Wall 3 | Plain concrete | Internal and external faces |
| Wall 4 | Concrete + water proofing admixture | None |

### 4.3 Test procedures

The test method followed a non-standard procedure simplified from procedures given in ASTM E514 (1985). Using a simple water jet set up, the corner of the test walls were sprayed at about 45$^{o}$ in the vertical plane and at a discharge rate of about 2.8 l/min to roughly simulate the average wind driven rain intensity on exposed wall. The walls' rain penetration performance was evaluated through visual observation. Photo shots were taken up to 90 minutes at 5 minutes intervals. Figure 2 shows a typical test wall setup.

## 5 RESULTS

### 5.1 Unplastered wall (plain concrete units)

Figures 3(a-c) show the observations on the internal face of the unplastered mortarless wall built from plain concrete interlocking block units. Damp patches started to develop after about 2 minutes of spraying and became more substantial after 5 minutes. After 30 minutes, almost the whole wall face was covered with damp patches. During the early stage, the damp patches came from the water that seeps through both the vertical and horizontal (bed) joints. Water seepage through the joints intensified causing the damp patches to spread over the wall face when exposed further to water spraying. Water penetration also came from the absorption through the block but at a much slower rate as compared to water penetration through the joints.

Figure 3(b) illustrates that the moisture stains always start where the vertical joint intersects the lower horizontal joint. The vertical joints clearly perform worse than the horizontal joints. This indicates that the interlocking feature at the vertical joint was not as effective as at the horizontal joint. The size the gap at the horizontal joint was much smaller (tighter) due to the block self weight. Water from the external face of the wall seeped through the vertical joint and accumulated at the bottom horizontal joint through gravity.

Block units from concrete with water proofing admixture which had a lower water absorption provided good resistance to water penetration. This was evident from the corner half-block units, which were not dampened as shown in Figure 3(c). The condition of the plain concrete unit and the admixture concrete unit after been exposed to water spraying is shown in Figures 4(a & b).

### 5.2 Wall plastered on internal face (plain concrete units)

Test observations on mortarless block wall internal plastering are shown in Figures 5(a-f). Internal plastering only seemed to delay the water penetration. The damp stains started to be visible after about 5 minutes of test. The extent of the damp stains at 60 minutes was about the same level of that for Wall 1 at 30 minutes. Water penetration came mostly through the joints. The damp patterns are quite similar to that of Wall 1.

### 5.3 Wall plastered on both faces (plain concrete units)

Observations on mortarless block wall plastered on both the internal and external faces being subjected to water spray are shown in Figures 6(a-d). The test showed that plastering on the external face of the block walls effectively resisted water from penetrating through the wall. The vertical damp stain observed was due to over-sprayed water that flowed down from top of the wall. Further examination on the wall section showed that water from outside penetrated through the plaster thickness only and did not affect the block unit (see Figure 6(d)).

### 5.4 Unplastered wall from blocks with admixture

The results of the rain simulation test on the wall constructed using blocks of concrete with water proofing admixture are shown in Figures 7(a-f). No wet spot was detected for the first 30 minutes of test. After that period, only three isolated wet spots developed at the horizontal joint of the corner units. These wet spots propagated slowly along the bed joint up to a maximum of about 150 mm long. Further examination shows that at the joint, only the surface of the outer shell was affected (Figure 7(e)). Unabsorbed water that seeped through the joints only managed to move half way through the block web and then drained down the sides of the core. Within the test period, the water spray had only wet the skin of the block units and was not absorbed into the block mass. The wet spots that developed at the corner blocks were due to water moving in through the vertical joints (Figure 7(f)).

## 6 CONCLUSIONS

1. The interlocking features for the block units under investigation did not provide a satisfactory resistance to rain penetration. The moisture stains start at the intersection between the vertical joint and the lower horizontal joint. The penetration was worse at the vertical joints than the horizontal joints

2. The addition of water proofing agent in the block making which decreased the water absorptiveness of the concrete block, consequently, improved the rain penetration resistance of the wall. Nevertheless, rain still penetrated the blockwork through the mortarless joints.

3. Proper plastering on the external face of the mortarless blockwork provided the necessary rain penetration resistance. Proper design and construction details as that for conventional masonry to drain off water collected at the bottom of the wall should be looked into.

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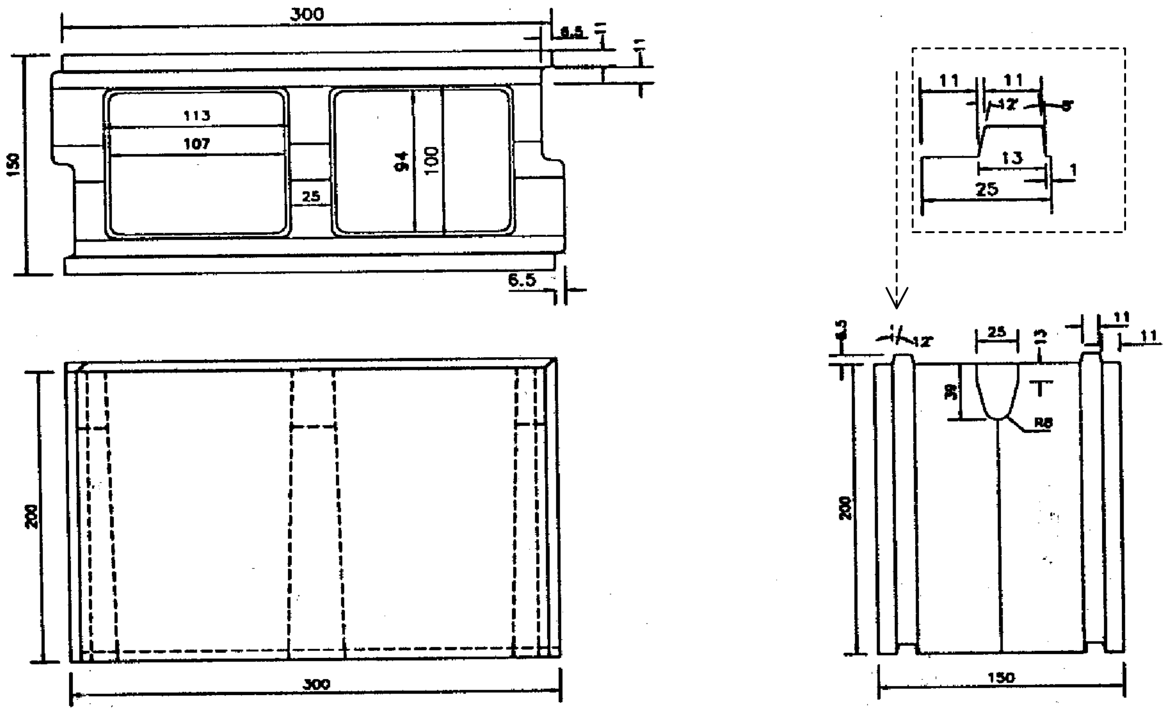
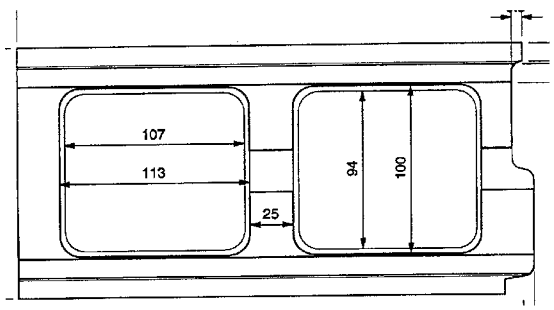



(a) Stretcher units (plan, front and side views)

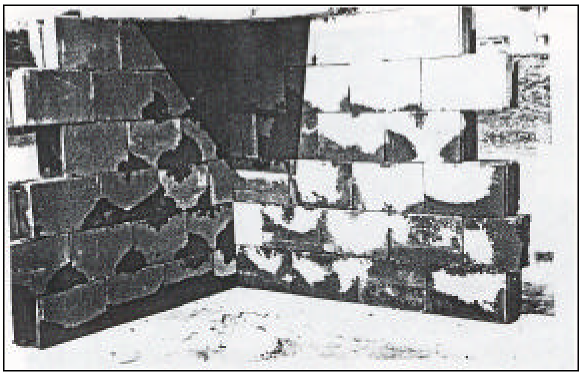



(b) Corner Unit (plan view)

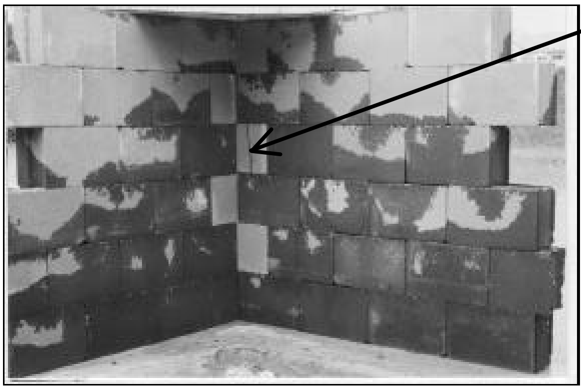
**Figure 1 . Details of the block units**

**Figure 2. The test arrangement**

(a) After 5 mins. spraying

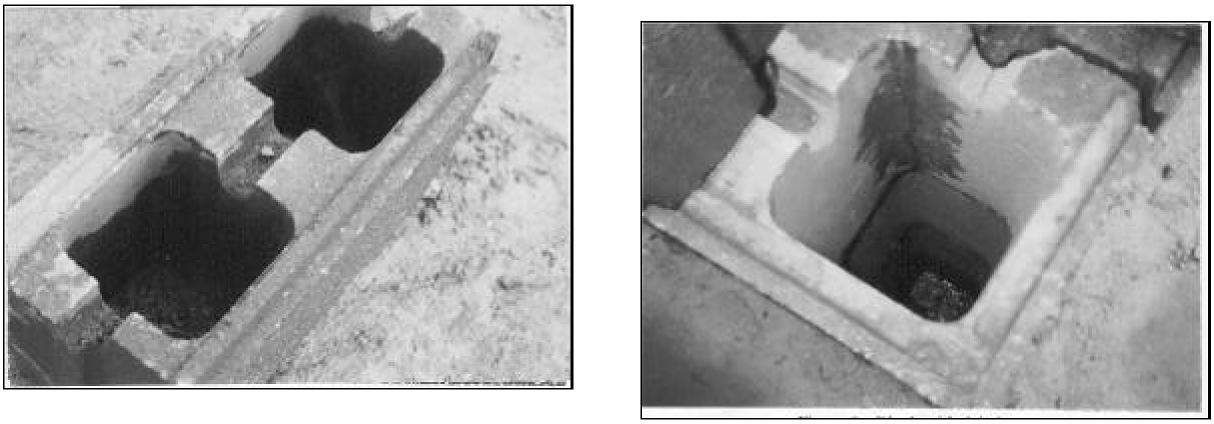


(b) After 10 mins. spraying



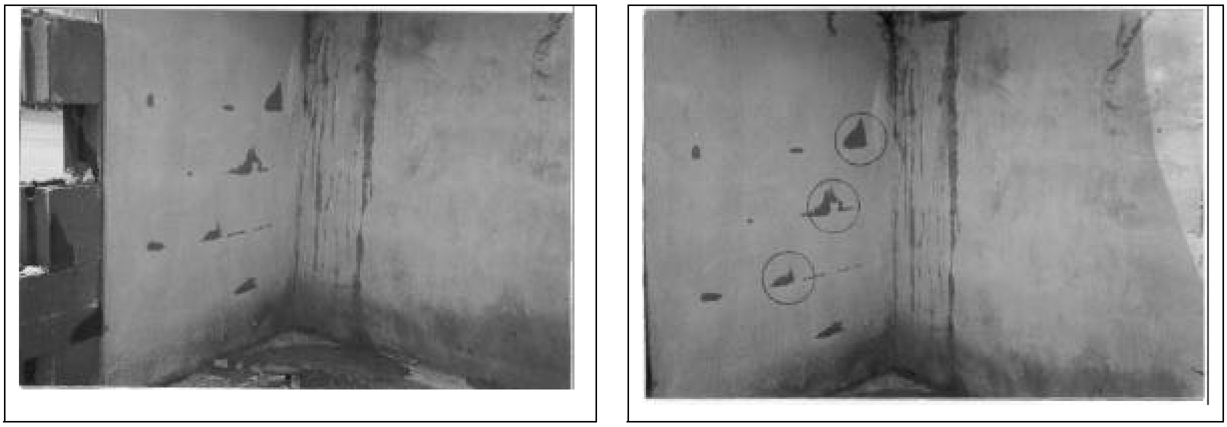

(c) After 30 mins. spraying

**Figure 3. Results of test wall 1 (unplastered wall)**

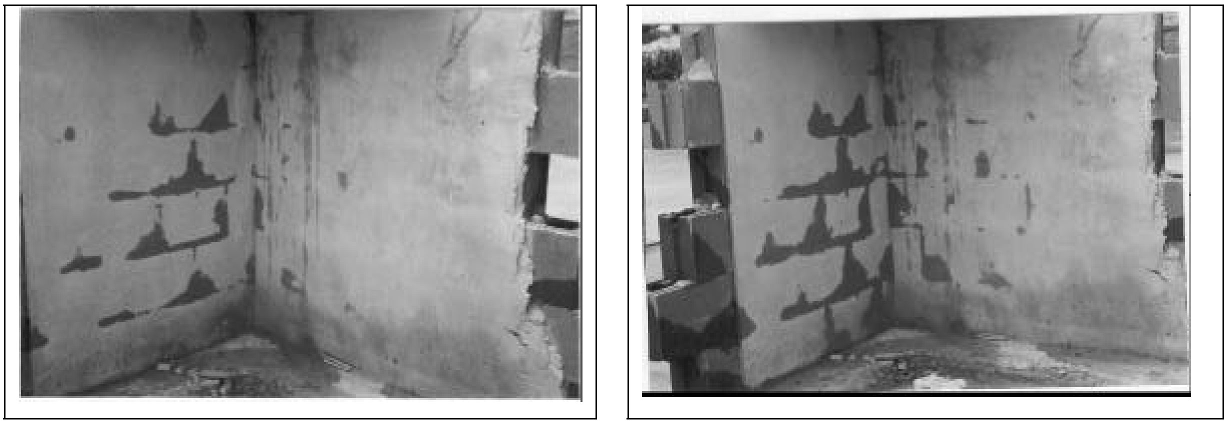


(a) Plain Concrete Unit (b) Unit From Concrete with Admixtures

**Figure 4. Water penetration through the block units**



(a) After 5 mins. Spraying (b) After 10 mins. spraying



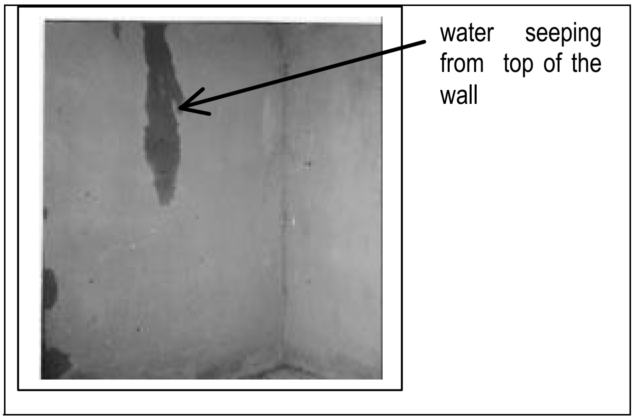
(c ) After 15 mins. Spraying (d)After 20 mins. spraying

(e) After 30 mins. Spraying

(f) After 60 mins. spraying

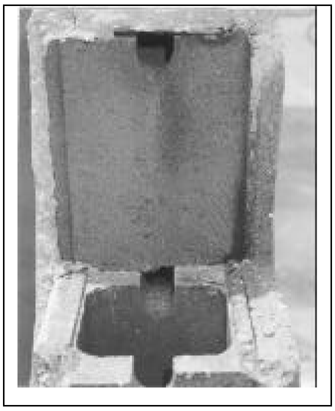
**Figure 5.– Observations on test wall 3 (plastered on internal face only)**

(a) After 5 mins. spraying




(b) After 60 mins. spraying

(c) After 90 mins. spraying



(d) Conditions of Unit and Internal Plaster

**Figure 6. Observations on test wall 4 (plastered on both faces)**

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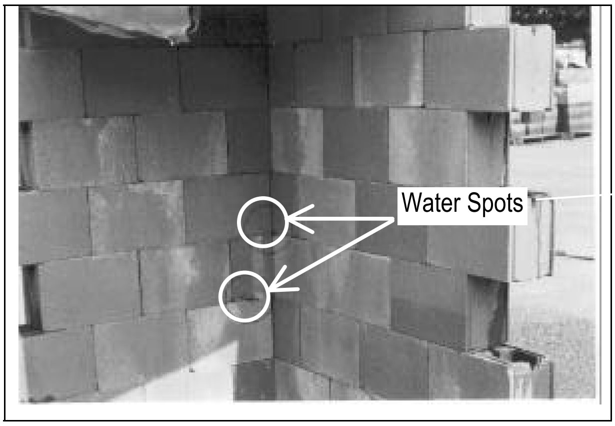
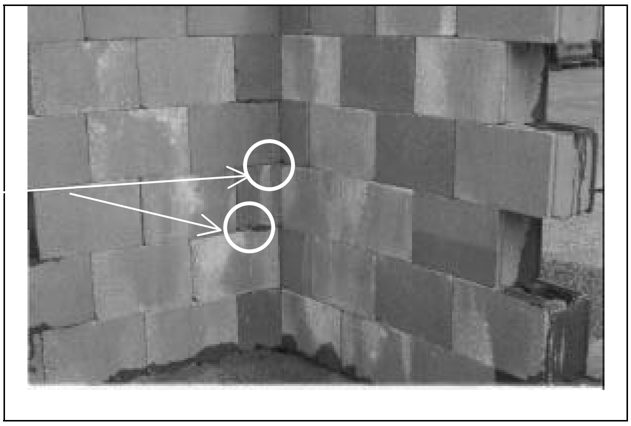
(a) After 5 mins. spraying

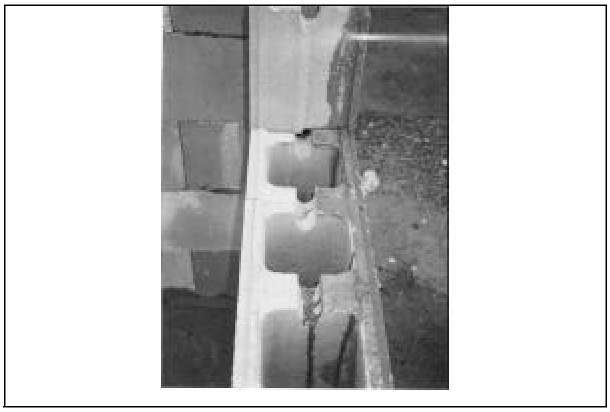
(b) After 10 mins. Spraying



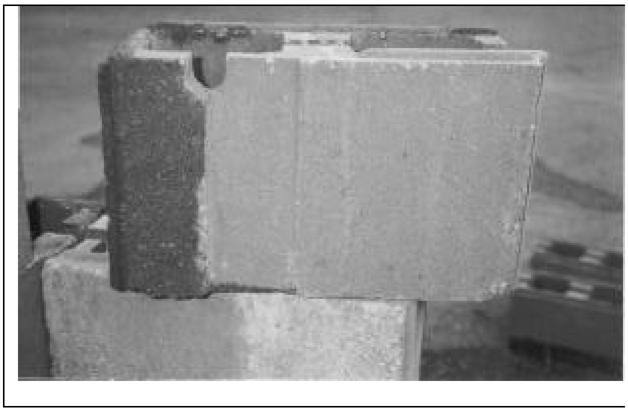

(c) After 30 mins. spraying



(d) After 90 mins. Spraying



(e) General condition of block units



(f) Condition of the corner unit

**Figure 7. Observations on test wall 5 (unplastered wall from units with water proofing admixture)**

# Optimum Fly Ash Content For Life-Cycle Cost

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**Summary: It has been established that fly ash concretes are durable in high chloride and high sulfate environments. Also, the addition of fly ash leads to improvement in concrete's resistance to alkali–silica reactivity. However, the decision to use fly ash concrete and the fly ash content is usually based on the cost of the concrete. Thus, a methodology has been developed to estimate the optimum fly ash content for the minimum concrete cost and maximum service life of the concrete. Concrete of different fly ash contents were prepared and their relative costs were calculated. Durability properties of some of the concrete were determined by experimental measurement of their relative service life. Optimum fly ash content was determined based on concrete cost and relative service life. Such a fly ash content can be considered as the fly ash content for minimum life-cycle cost.**

**Keywords: Concrete cost, service life, fly ash content, life-cycle cost**.

## 1 INTRODUCTION

Extensive studies have been carried out on fly ash concretes and their improved durability has been very well established. However, in many commercial developments, the use of fly ash concrete and the fly ash content is selected on purely the concrete cost. The cost of the concrete is only one of many parameters governing life-cycle cost. The fly ash content should be selected based on concrete cost and service life. Service life is critical for the owner of a structure, as the longer the structure is in service, the more 'value' the owner derives from the structure. From the owner's perspective, the total life-cycle cost instead of the initial cost of building the structure is more important.

Concretes with a range of fly ash contents were prepared and their costs were evaluated. The basis of comparison was similar 28-day compressive strength because the concrete is normally specified by its 28-day strength. Relative service life was determined experimentally.

Firstly, the fly ash content was determined for the minimum concrete cost. Secondly, the fly ash content was determined for maximum service life. Thirdly, the optimum fly ash content was determined for both cost and service life. By using this recommended methodology, concrete can be selected which has the minimum total life-cycle cost.

## 2 EXPERIMENTAL PROCEDURES

### 2.1 Fly ash content for minimum concrete cost

A range of concrete mixtures were prepared from normal Portland cement (NPC) and a fly ash (FA). The fly ash used for the concrete was a class F fly ash. The total cementitious material and fly ash contents were varied, and the 28-day compressive strength was determined for each mixture. The workability, measured by slump, was kept constant. A water-reducing agent was used at a fixed dosage of 400 ml/100 kg of cementitious material.

A strength margin of 7 MPa was used to determine the mean strength for each strength grade. The relationship between the total cementitious material content and the nominal fly ash content was established for Grades 20, 25, 32, 40 and 50 concretes, and is shown in Fig. 1. Once the total cementitious material content and fly ash content has been determined for the various grades, the cost of the concrete materials was calculated, and is shown in Table 1 and plotted in Fig. 2. The relative cost in Fig. 2 is the cost of the cementitious material and chemical admixture, and is based on the prices of materials in Sydney, Australia, and the prices are: $160 per tonne for NPC, $50 per tonne for fly ash and $0.8 per litre for the chemical admixture. The cost of aggregates will be very similar and has not been accounted for in the relative cost.

The method adopted in this study is similar to that reported by Butler (1988) except that all concrete mixtures contained a water-reducing admixture, reflecting the common type of concrete used in the industry in Australia at present.

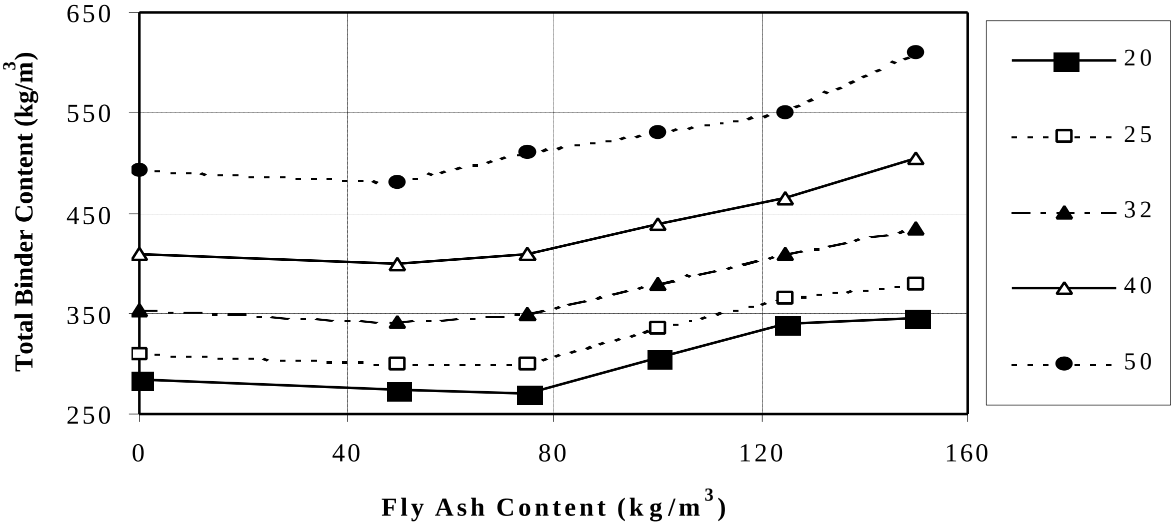


**Figure 1. Variation of total cementitious material content with fly ash content for the various grades. The legend shows the grade of the concretes.**

**Table 1. Major cost of the concretes (A$/m$^3$) with different fly ash content**

| *Fly ash content (kg/m$^3$)* | *Strength grade (MPa)* | | | | |
|---|---|---|---|---|---|
| | *20* | *25* | *32* | *40* | *50* |
| 0 | 47.6 | 52.14 | 57.61 | 68.96 | 82.92 |
| 50 | 40.17 | 44.71 | 50.31 | 61.53 | 74.99 |
| 75 | 36.79 | 41.84 | 48.87 | 60.34 | 77.16 |
| 100 | 39.80 | 44.85 | 51.02 | 62.51 | 77.65 |
| 125 | 42.81 | 47.02 | 53.16 | 63.84 | 78.14 |
| 150 | 40.78 | 46.67 | 54.49 | 67.69 | 85.35 |

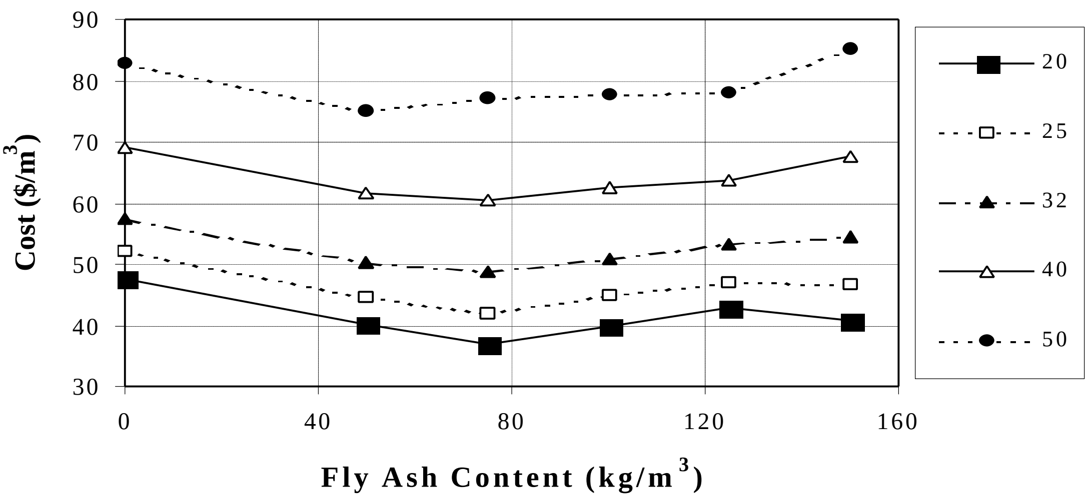


**Figure 2. Cost of cementitious material(s) and water-reducing admixtures versus fly ash content for various grades of concrete. The legend shows the grade of the concretes**.

### 2.2 Fly ash content for maximum service life

#### 2.2.1 Chloride resistance

Ponding tests were conducted on concrete samples prepared from an NPC and also prepared with 15% and 30% by weight of cementitious material as fly ash. The samples were immersed in 3% NaCl solution for a period of 33 months and subsequently the samples were drilled and powdered samples were obtained at various depths. The powdered samples were analysed for water-soluble chloride concentrations by ion chromatography to establish the chloride profile. These chloride profiles were subsequently used to calculate apparent diffusion coefficient using Fick's second law.

2.2.2 Sulfate resistance

Sulfate resistance of three cementitious materials was evaluated and the three cementitious materials were NPC, NPC with 15% by weight of fly ash and NPC with 30% by weight of fly ash. The resistance to sulfate of these cementitious materials were tested according to ASTM C 1012. Expansion of mortar prisms was measured after immersion in 5% $Na_2SO_4$.

# 3 RESULTS

## 3.1 Fly ash content for minimum concrete cost

It is clear from Fig. 2 that the fly ash content for minimum concrete cost appears to be in the region of 50–75 kg/$m^3$. Increasing or decreasing the fly ash content leads to an increase in the cost of the concrete. The cost of various grades of concrete for the optimum fly ash content was calculated and is shown in Fig. 3. Figure 3 also shows the optimum fly ash content as the percentage of the total cementitious material content. As expected, the cost of FA concrete increased as the grade of the concrete was increased. The optimum fly ash content was found to decrease with the grade of concrete. For a Grade 20 concrete, the fly ash content for the minimum concrete cost is about 28% of the total cementitious material content, whereas for a Grade 50 concrete, the fly ash content for the minimum concrete cost is only about 10% of the total cementitious material content.

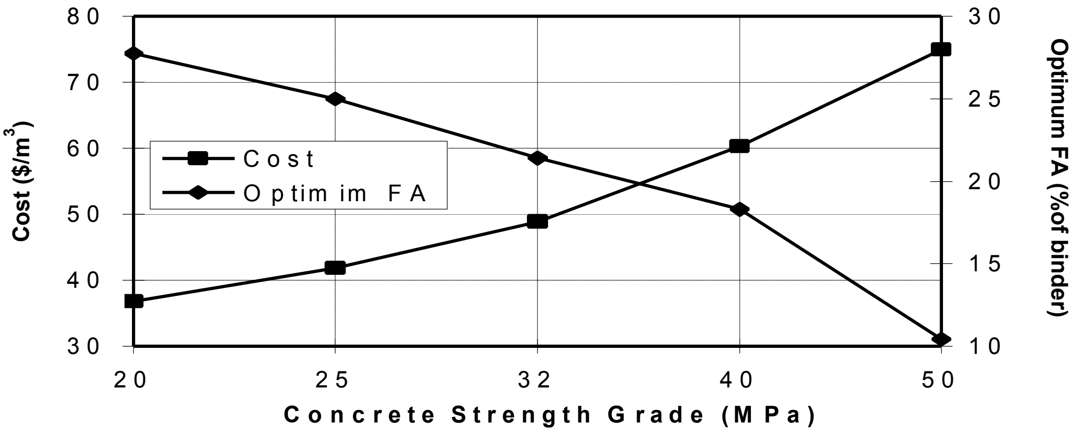


**Figure 3. Optimum fly ash content and major cost of concrete for various cementitious materials.**

## 3.2 Fly ash content for maximum service life

3.2.1 Chloride resistance

Table 2 gives the details of concrete mixes and the apparent diffusion coefficients of NPC and FA concretes. It is clear from the table that a 15% addition of fly ash leads to a significant reduction in the apparent diffusion coefficient, and increasing the fly ash content from 15% to 30% leads to a further reduction in the apparent diffusion coefficient. Some other studies have also shown that increasing the fly ash content up to 40% of the total cementitious material content leads to improvement in the durability of concrete to a high chloride environment (Mangat & Molloy 1994; Schießl & Wiens 1997; Cao *et al.* 1994). Thus, increasing the fly ash content leads to improvement in the resistance to a chloride environment.

3.2.2 Sulfate resistance

Figure 4 shows the expansion of mortar prisms prepared from the three binders, namely NPC, NPC with 15% FA and NPC with 30% FA. It can be seen in Fig. 4 that the expansion observed in mortar prisms decreases with an increase in the fly ash content. Thus, a fly ash concrete is expected to have significantly superior resistance in a sulfate environment, and also increasing the fly ash content leads to further improvement in the resistance. A number of other studies also confirm that the inclusion of fly ash improves the sulfate resistance, and increasing the content up to 40% of the total cementitious material leads to improvement in the durability of concrete to a high sulfate environment (Tikalsky & Carasquillo 1992; Malhotra et al. 1991; Sirivivatnanon et al. 1995).

Thus, for both chloride and sulfate environments, a fly ash content of approximately 40% would give the best resistance to both aggressive environments.

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**Table 2 – Apparent diffusion coefficient of NPC and FA concretes.**

| *Binder* | *Water/Binder* | *28-day strength (MPa)* | *Apparent diffusion coefficient ($m^2$/sec)* |
|---|---|---|---|
| 100% NPC | 0.44 | 52.0 | $79.2 \times 10^{-12}$ |
| 85% NPC + 15% FA | 0.46 | 52.0 | $1.13 \times 10^{-12}$ |
| 70% NPC + 30% FA | 0.37 | 54.0 | $0.51 \times 10^{-12}$ |

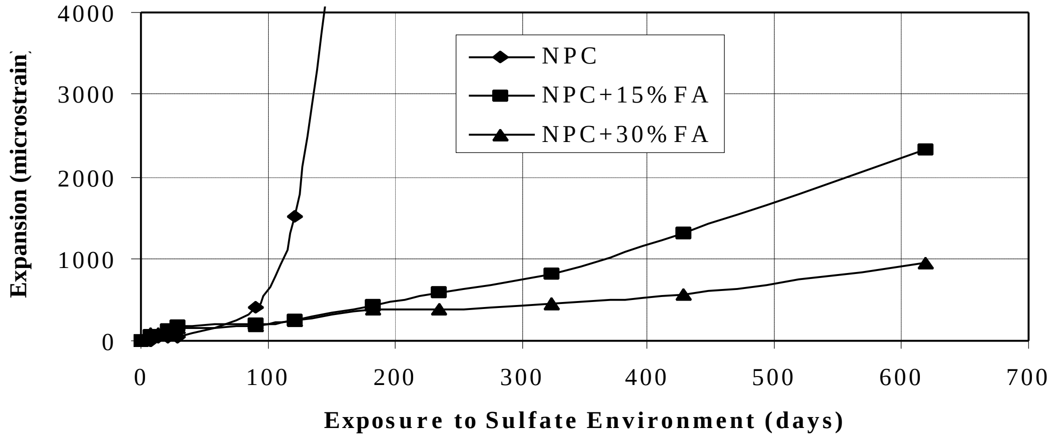


**Figure 4. – Expansion of mortar bars on exposure to 5% $Na_2SO_4$ measured according to the procedure of ASTM C 1012.**

### 3.3 Optimum fly ash content for life-cycle cost

After establishing the fly ash content for the minimum concrete cost and for maximum service life, the optimum fly ash content is established for life-cycle cost. The fly ash content for minimum concrete cost ranges from 28% for Grade 20 concrete to 10% for Grade 50 concrete, whereas the fly ash content for maximum service life in aggressive chloride and sulfate environments is 40%. To resolve the anomaly between the two fly ash contents, attempts were made to determine fly ash content, which gives both lower concrete cost and higher service life. In other words, fly ash content was evaluated for minimum life-cycle cost.

To evaluate the fly ash content to achieve minimum life-cycle cost, it is essential that both concrete cost and service life can be quantified for various grades of concrete. The relative cost of the concrete has already been calculated. In the absence of quantified service life of concrete in sulfate environments, the optimum fly ash content for life-cycle cost in sulfate environments was not evaluated. Optimum fly ash content for life-cycle cost of concrete in chloride environments was evaluated.

Service life for concrete in chloride environments was experimentally measured on concrete slab samples. These 300 $mm^2$ slab samples represent small section of a structural element typically used in various concrete structures. An F81 (8 mm bars and 100 mm centres) mesh was used as reinforcement for the slabs. The samples were partially immersed to a depth of 40 mm in a 3% sodium chloride solution. Chloride ion penetration into concrete slab samples was mainly achieved by upward diffusion via the capillary pores. The evaporation of water through large areas of slab exposed to air was expected to accelerate the movement of chloride ions. Such an acceleration of movement of chloride ions is necessary to expedite the corrosion, and thus the samples have shortened service lives which are practical to measure. Hence, these experimentally measured service lives were expected to be different to actual service life in the field, but they should be very useful for the purpose of comparison between different cementitious materials. Also, since these service lives are different to actual service lives in the field, they are appropriately termed ‘relative service lives’. The details of the measurement of relative service life are given elsewhere (Sirivivatnanon *et al.* 2000).

Table 3 gives the relative service lives of concretes of nominal Grades 20, 40 and 50. The study was carried out on three grades of NPC concrete and a 30% fly ash concrete. Ideally relative service lives should be established for a range of fly ash contents, and a table similar to Table 1 should be prepared for relative service lives. Such a table would be essential to determine the optimum fly ash content for both durability, as reflected by relative service life, and concrete cost.

**Table 3. Relative service life of concretes in days**

| *Fly ash content* | | *Strength grade (MPa)* | | |
|---|---|---|---|---|
| *kg/m³* | *Per cent by weight of total cementitious material* | *20* | *40* | *50* |
| 0 | 0 | 71 | 356 | 484 |
| 90 | 30.0 | 624 | — | — |
| 115 | 30.0 | — | 994 | — |
| 135 | 30.0 | — | — | 1414 |

Equation 1 gives the service life ($t$ in seconds) as calculated by Fick's second law. It can be seen that the service life is inversely proportional to the diffusion coefficient ($D_a$ in m$^2$/sec).

$$t = \frac{x^2}{4D_a}\left(erf^{-1}\left[\frac{C_t(x,t) \; - C_s}{C_s}\right]\right)^{-2} \qquad (1)$$

where $x$ is the cover depth (m), $C_t(x,t)$ is the chloride concentration at cover depth (% by mass of concrete), $C_s$ is the surface chloride concentration at time $t$ (% by mass of concrete), and $erf^{-1}$ is the inverse of error function.

The inverse of apparent diffusion coefficient values given in Table 2 were plotted with respect to the fly ash content and are shown in Fig. 5. The trend of Fig. 5 was used to interpolate relative service life values for various fly ash contents, which are shown in Table 4.

The cost of concretes with various fly ash contents is given in Table 1 and the relative service lives are given in Table 3. Ideally the fly ash content is to be evaluated which gives the minimum concrete cost and maximum relative service life. Thus, the fly ash content which gives the minimum concrete cost per unit of relative service life should be the optimum fly ash content based on both concrete cost and service life. Table 5 and Fig. 6 give the ratio of concrete cost to service life for a range of fly ash contents. This ratio incorporates the concept of 'life-cycle costing' and can be interpreted as the cost of the concrete per unit time of its service. Essentially the ratio is the 'true' cost of the concrete and is an indication of how much the concrete will cost for a specified duration. The true life-cycle costing will also include the interest accrued on the initial expenditure and the maintenance costs, but the ratio in Fig. 6 is a good approximation of the life-cycle cost.

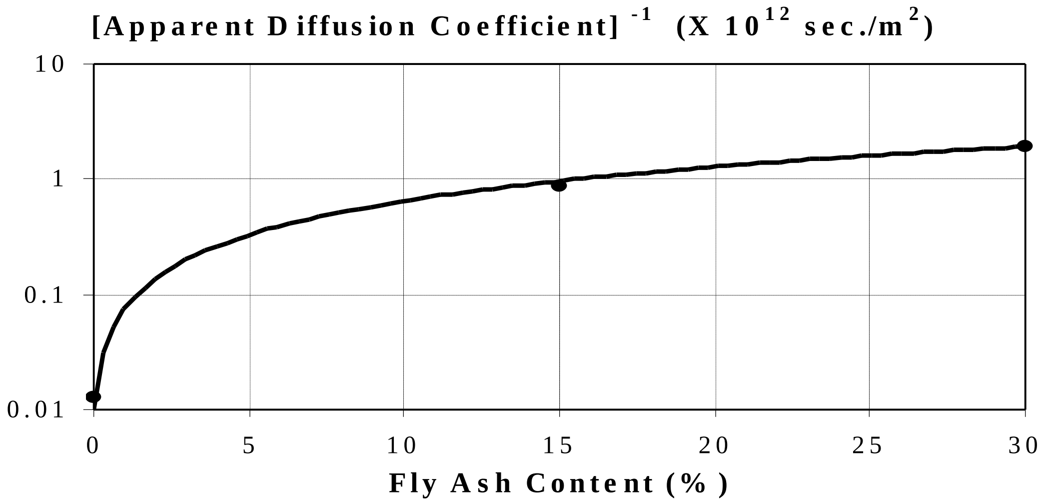


**Figure 5. Inverse of apparent diffusion coefficients of an NPC concrete and two fly ash concretes.**

**Table 4. Interpolated value of relative service life in days**

| *Fly ash content (kg/m³)* | *Strength grade (MPa)* | | |
|---|---|---|---|
| | *20* | *40* | *50* |
| 0 | 71 | 356 | 484 |
| 50 | 424 | 637 | 820 |
| 75 | 594 | 777 | 988 |
| 100 | 764 | 918 | 1156 |
| 125 | 935 | 1059 | 1324 |
| 150 | 1105 | 1200 | 1492 |

**Table 5. Ratio of concrete cost to relative service life**

| *Fly ash content ($kg/m^3$)* | *Strength grade (MPa)* | | |
|---|---|---|---|
| | *20* | *40* | *50* |
| 0 | 0.584 | 0.174 | 0.150 |
| 50 | 0.097 | 0.094 | 0.084 |
| 75 | 0.067 | 0.075 | 0.068 |
| 100 | 0.050 | 0.062 | 0.057 |
| 125 | 0.039 | 0.052 | 0.049 |
| 150 | 0.032 | 0.045 | 0.043 |

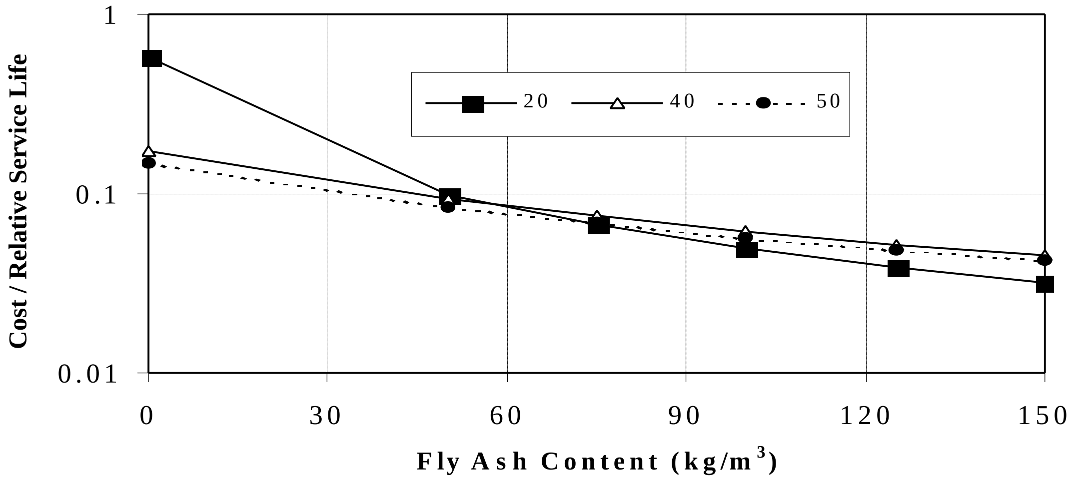


**Figure 6. Variation of the ratio (concrete cost/relative service life) with fly ash content.**

It can be seen from Fig. 6 that the ratio of concrete cost to relative service life is minimum for the maximum fly ash content for all the three grades studied. The concrete cost is minimum for fly ash content of about 50–75 $kg/m^3$ and the concrete cost for higher fly ash content is only marginally higher. However, the relative service life significantly increases with an increase in fly ash content, and thus the ratio of concrete cost to relative service life also decreases with an increase in fly ash content. It is anticipated that a further increase in fly ash content beyond 150 $kg/m^3$ may also lead to a decrease in the ratio of concrete cost to relative service life, but due to unavailability of any data, it is difficult to pronounce this with confidence.

Thus, the optimum fly ash content based on both concrete cost and service life is the maximum fly ash content. It should be noted that the fly ash content for maximum service life is the same as the optimum fly ash content based on both concrete cost and service life.

## 4 CONCLUSIONS

1. For Grade 20 concrete, the fly ash content for the minimum concrete cost is about 28% of the total cementitious material content, whereas for Grade 50 concrete, the fly ash content for the minimum concrete cost is only about 10% of the total cementitious material content.
2. For both chloride and sulfate environments, a fly ash content of 40% would give the best resistance to both aggressive environments.
3. The optimum fly ash content based on both concrete cost and service life is the same as that based on service life only. In both cases the optimum fly ash content is the maximum fly ash percentage.
4. A methodology has been developed which gives the optimum fly ash content for concrete, which has the minimum life-cycle cost.

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# Monitoring The Corrosion Of Concrete Reinforcement Using Control Curves

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**Summary: Corrosion testing (half-cell and LPR) was carried out on a number reinforced concrete panels which had been taken from the fascia of a twenty five year old high rise building in Melbourne, Australia. Corrosion, predominantly as a result of carbonation of the concrete, was associated with a limited amount of cracking. A monitoring technique was established in which probe electrodes (reference and counter) were retro-fitted into the concrete. The probe electrode setup was identical for all panels tested. It was found that the corrosion behaviour of all panels tested closely fitted a family of results when the corrosion potential is plotted against the polarisation resistance ($R_p$). This enabled the development of a so-called 'control curve' relating the corrosion potential to the $R_p$ for all of the panels under investigation. This relationship was also confirmed on laboratory samples, indicating that for a fixed geometry and experimental conditions a relationship between the potential and polarisation resistance of steel can be established for the steel-concrete system. Experimental results will be presented which indicate that for a given monitoring cell geometry, it may be possible to propose criteria for the point at which remediation measures should be considered. The establishment of such a control curve has enabled the development of a powerful monitoring tool for the assessment of a number of proposed corrosion remediation techniques. The actual effect of any corrosion remediation technique becomes clearly apparent via the type and magnitude of deviation of post remediation data from the original (pre-remediation) control curve.**

**Keywords: Corrosion in concrete, Monitoring, Reinforcing Steel, Polarisation Resistance, Control Curve**

## 1 INTRODUCTION

Reinforced concrete is undoubtedly the major material of construction in the world today, thus the corrosion of concrete reinforcement is considered a major issue facing the civil engineering sector in modern society. Corrosion of reinforcing bars can not only lead to costly remediation, but may end the life of a structure well prior to its design life, causing massive financial disruptions in cases where structures are both costly and provide a source of revenue (e.g. high rise buildings, jetties, etc.). Most developed cities presently have a large number of buildings whose facades consist of reinforced concrete fascia panels suspended over the face of the building. Although the manufacture of such panels can be carried out under controllable off-site conditions resulting in a good quality product, when buildings have a design life from 25 to 100 years, deterioration of the panels can eventually take place. Carbonation of the concrete by atmospheric $CO_2$, sulphate attack from urban pollution or chloride ingress through cracks can lead to cases where corrosion of the reinforcement can occur, ultimately leading to spalling of the concrete. The consequences of such an event are such that responsible maintenance of a building requires routine monitoring of the possibility of reinforcement corrosion.

In the event that corrosion of the concrete reinforcement occurs, several methods are presently available for the remediation of the condition and the extension of life of the structure. These methods include, cathodic protection (CP), re-alkalisation, desalination (electrochemical chloride removal), crack sealing, anti-carbonation or anti-moisture coatings and inhibitors. Most often the selection of an appropriate remediation technique will depend on factors that are unique to each individual situation.

The ability to monitor the corrosion of steel that can be buried several centimeters beneath concrete presents certain challenges, especially in terms of physical inspections. Clearly, suitable reinforcement corrosion monitoring techniques must be non-destructive, whilst also providing accurate information regarding the corrosion activity of embedded steel. Techniques that have been developed to monitor corrosion processes that may be occuring at the reinforcement surface, but of which there is as yet no evidence on the concrete surface, include: half cell potential measurements, concrete resistivity measurements (Gerritsen and Dacre, 2001) and linear polarisation resistance measurements. Each of these techniques provides some information as to what is going on within the concrete, but none give the complete story. Indeed recently much scrutiny has been focused on the accuracy and suitability of these common monitoring techniques, namely LPR, for use on the steel-concrete system (Birbilis et al, 2001). However, this paper introduces an alternative approach to analysing data such that conventional monitoring techniques may become more meaningful, especially in the context of ranking corrosion remediation techniques.

## 2 HALF CELL POTENTIALS

Measuring the corrosion potential (half-cell potential, HCP) of reinforcing bars is the commonest of the corrosion monitoring methods available, and is widely used in practice as an indication of the possible corrosion risk. HCP monitoring is outlined in ASTM C876. “Standard Method for Half Cell Potentials of Uncoated Reinforcing Steels in Concrete” (1993). In this technique the electrical potential of the reinforcement is measured against a reference electrode (half-cell) placed on the surface of the concrete immediately above the reinforcement. The non-mandatory information appended to the Standard reports an empirical interpretation of the results obtained. This interpretation is summarised in Table 1 below:

**Table 1. ASTM C876 Criteria (1991)**

| *Half Cell Potential (vs. Cu/CuSO$_4$ electrode)* | *Corrosion Condition* |
|---|---|
| > -200mV | Less than 10% probability of corrosion |
| -200mV > -350mV | Between 10% and 90% probability of corrosion |
| < -350mV | Greater than 90% probability of corrosion |

The corrosion potential however only indicates the relative risk of corrosion, providing no quantitative information regarding corrosion rates. This indication of corrosion risk by the corrosion potential reading may be based upon a simple model of the quality of passive film that protects the steel in good quality concrete. The quality of the passive oxide film upon the steel may be represented by α, where α may represent the ratio of filmed to unfilmed area in the corrosion microcell upon the steel surface. In the case where diffusion effects upon the corrosion potential can be neglected, it is shown by Cherry (2001) that the corrosion potential can be represented as:

$$E_{corr} = \ln a - K - \frac{I_{corr}\, r}{B} \qquad (1)$$

where $B = \dfrac{b_a b_c}{2.3(b_c + b_a)}$ and $K = \dfrac{E^a{}_o}{E^c{}_o}$

$E_{corr}$ is the corrosion potential, $I_{corr}$ is the corrosion current, $ß_a$ and $ß_c$ represent the Tafel slopes for the anodic and cathodic processes, ? represents the electrical resistance associated with the anodic portion of the corrosion circuit and $E^a{}_o$ and $E^c{}_o$ represent the equilibrium electrode potentials of the anodic and cathodic processes respectively.

Equation 1 gives a qualitative rationalisation of the ASTM C876 criteria, in that it indicates an increase in the corrosion current as the corrosion potential becomes more negative. Although this relationship is generally observed, it has been shown by Cherry *et al* (1986) that corrosion potentials measured at the surface of the concrete will vary markedly with the thickness and resistivity of the concrete. This serves as somewhat of a limitation to the HCP technique.

It is also possible that corrosion potentials can provide misleading information when used with some more recently developed remediation measures. For example, effective inhibitors may have a significant impact on the exchange current densities of either cathodic or anodic corrosion reactions. In such cases the relative movement of $E_{corr}$ may give no indication of the effectiveness of the inhibitor when using ASTM C876. Anti-carbonation coatings and silane type concrete coatings may affect the resistance polarisation of the cathodic or anodic corrosion processes by varying amounts and so again, a reduction in corrosion current may be accompanied by either positive or negative movements of the rest potential. As a result a more independent estimate of the corrosion current is required when assessng corrosion remediation measures.

## 3 LINEAR POLARISATION RESISTANCE

To date, the most common method of monitoring the corrosion rate of concrete reinforcement is based on the Linear Polarisation Resistance (LPR) technique as developed for metals in solution in the late 1950’s. In the LPR technique, the potential of the rebar is scanned slowly over a small range in the vicinity of the open-circuit (corrosion) potential. As the current densities and potential shifts associated with the technique are small, it was considered in the original derivation of the theory that mass transport (diffusion) effects could be neglected. The slope of the polarisation curve (i.e. the potential change divided by the applied current change) at the point at which the applied current equals zero is defined as $R_p$, or the Polarisation Resistance, and is considered a measure of the opposition to metallic dissolution.

If we assume that the total area of the corroding electrode (reinforcement) probed by an LPR measurement is $s$, then $R_p$ can be converted to a corrosion current rate via:

Where B has been previously defined, and is reported to have values between 14 and 52mV for the steel in concrete system.

$$I_{corr} = \frac{B}{s R_p} \qquad (2)$$

Both the values of B and $s$ are somewhat uncertain, thus equation 2 cannot give accurate figures for the rate of corrosion. It has been highlighted by Roberge (1999) that the theoretical considerations used to derive LPR theory, strictly speaking, rarely apply under actual operating conditions. None the less it is shown in this paper that there is considerable value in monitoring with LPR, especially for determining relative corrosion rates brought upon by environmental changes in samples with fixed geometry.

It has been reported by Cherry (2001) that the critical corrosion current density of $1mA/m^2$ is considered by various authors to be the borderline corrosion rate for passive steel in concrete.

## 4 EXPERIMENTAL

A number of precast reinforced concrete fascia panels were obtained. These panels were 75mm thick and reinforced along their centre line by a mesh of 8mm∅ reinforcement. Corrosion probes of the form indicated in Fig. 1 were installed (retro-fit) into the panels just above an intersection point of the reinforcing mesh. The probes installed consisted of a silver/silver chloride (saturated) reference electrode mounted such that its tip was about 20mm from the reinforcing bar, and two mixed metal oxide (MMO) coated titanium rods that served as counter electrodes that could provide a polarising current to the reinforcing bar to the concrete.

The corrosion potential was measured using the reference electrode, whilst sufficient current was then provided by the counter electrodes to the reinforcing bar to shift the corrosion potential in the cathodic (more negative) direction by no more than 20mV. A plot of the impressed current against the potential shift was used to yield values for $R_p$. In all cases a portable 3LP device was used for the measurements.

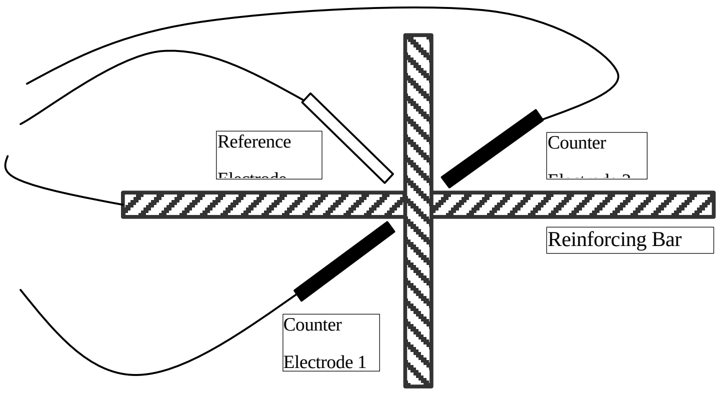


**Figure 1. Diagrammatic representation of polarisation cell**

Following installation of the probes, all panels were subjected to various wet-dry cycles in order to stimulate and suppress corrosion such that a spectrum of results could be obtained for panels of varying corrosion activities. The panels were made from very high quality concrete and were initially in a non-corroding state. Consequently the panels were all brought into a state of corrosion by means of electrochemical polarisation. Following on from this, various remediation techniques were applied in order to assess the relative effectiveness of the chosen techniques.

Data was also collected on reinforced mortar prisms specifically designed for laboratory testing. The prisms were 220×100×70mm with a 12mm∅ carbon steel reinforcing rod placed centrally in the longitudinal direction. The mortar mix was 4:2:1 (sand:OPcement:water). An Ag/AgCl reference electrode and a mixed metal oxide (MMO) coated titanium counter electrode were embedded in each prism. The prisms were exposed to various environments within the laboratory and electrochemical testing was carried out using a PAR/BES potentiostat.

## 5 RESULTS AND DISCUSSION

In view of the two sets of criteria for the detection of corrosion (ASTM C876 and current density of $1mA/m^2$) it was decided to determine the variation in $E_{corr}$ with the polarisation resistance for the fascia panels. A set of results is shown below in Fig. 2.

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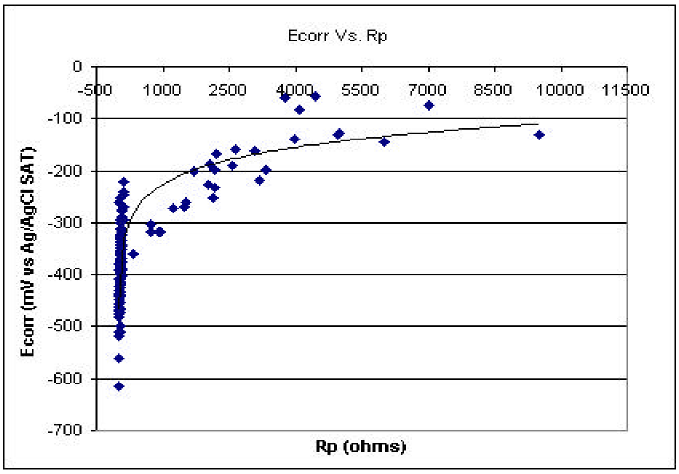


**Figure 2. Variation of polarisation resistance with half cell potential**

It should be noted that the results presented in Fig. 2 are reported with respect to a silver/silver chloride (saturated electrode) for which potentials of -150mV and -300mV correspond to -200mV and 350mV on the copper/copper sulphate scale. The results shown indicate a very apparent trend in the data, in that all results obtained (from twelve distinct monitoring locations that went into the construction of Fig. 2) seem to fit a 'family' of results. This trend becomes even more apparent when a log scale is used for the polarisation resistance values as shown below in Fig. 3.

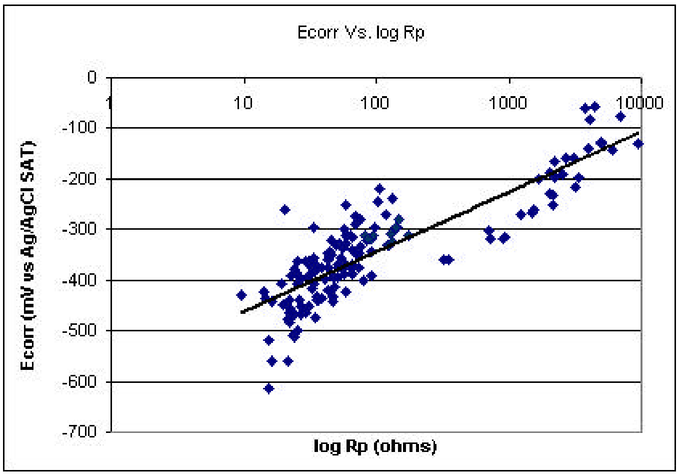


**Figure 3. Variation of log polarisation resistance with half cell potential**

The above results indicate that it is possible to define a distinct relationship between $E_{corr}$ and $R_p$, provided that the cell geometry of the samples under investigation remains constant. The development of a plot such as that seen in Fig. 3 can then indicate the value of $E_{corr}$ that corresponds to a corrosion current density of 1mA/m$^2$. In this case the geometry of the polarisation cell was constructed such that 100cm$^2$ of reinforcing bar is polarised, therefore putting B equal to 30mV in equation 2 indicates that a corrosion current density 1mA/m$^2$ corresponds to a corrosion potential of approximately -150mV. In this case there seems to be a good consistency between the results obtained using LPR and ASTM C876. However the construction of a control curve such as that seen in Fig. 3 can alleviate the use of ASTM C876 since customised criteria can be established for each unique situation under investigation. The criteria as set out in the ASTM standard are relevant for bridge decks in North America that are subjected to de-icing salts, however the majority of local reinforcement corrosion cases are destined to be subjected to conditions which bear little resemblance to such a situation. This notion can be confirmed when we inspect the control curve seen in Fig. 4. The data in Fig. 4 are from laboratory prism specimens that also have a polarised area of reinforcement equal to 100cm$^2$. In this case it is thus seen that a corrosion current density 1mA/m$^2$ corresponds to a corrosion potential of approximately -220mV, which is rather different to that of the fascia panels. This highlights the dependence of the control curve upon geometry of the polarisation cell.

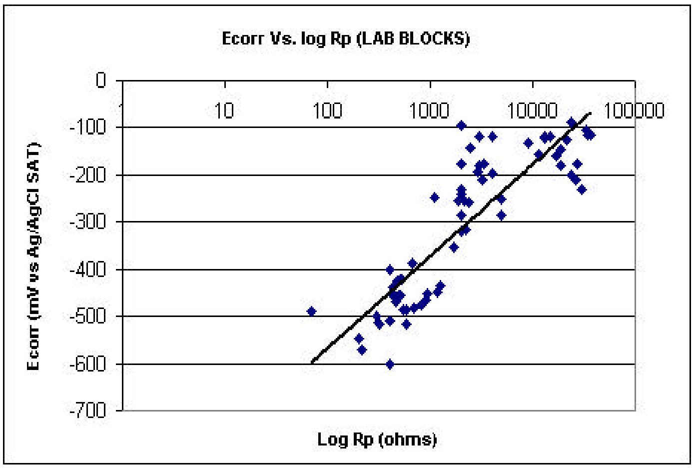


**Figure 4. Variation of log polarisation resistance with half cell potential for laboratory samples**

The notion of using control curves, such as those presented above, to monitor the corrosion of concrete reinforcement presents several advantages. The major reason for this is that measurements of the corrosion potential and polarisation resistance versus time, which is common practice within the industry, can be misleading due to environmental factors that can give results a cyclic nature. This phenomenon can be seen in Fig. 5 for data that was collected for a large marine structure over a period of seven months.

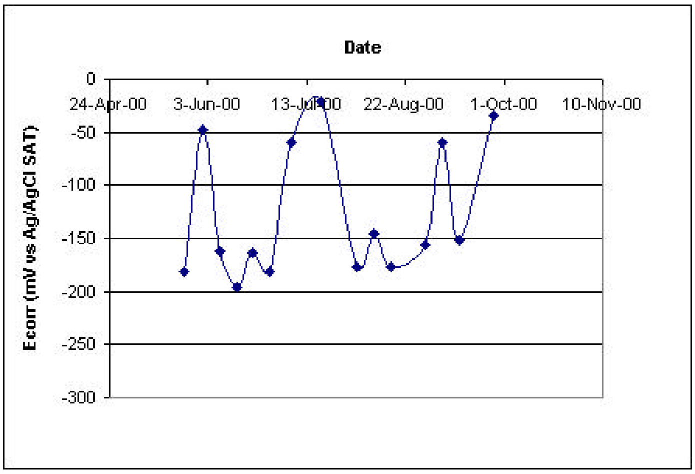


**Figure 5. Variation of corrosion potential with time for a marine structure**

When monitoring with control curves ($E_{corr}$ vs. $R_p$), the data obtained is combined into a 'family' of results that characterise the system under investigation. If monitoring occurs over a long period of time, then data can be added to the control curve in a time-dependant manner such that it becomes obvious whether or not the form (eg. gradient) of the control curve remains constant throughout the lifetime of a structure, or whether or not the control curve changes as a result of environmental factors that may be altering (i.e. properties of the concrete).

Once a control curve for a certain system has been established, such as that seen in Fig. 3, then this curve can act as tool for monitoring the effectiveness of any subsequent corrosion remediation techniques. The reason for this is that the type and magnitude of any deviation from the control curve of post-remediation data can be compared with that of pre-remediation data.

In order to assess the effectiveness of certain corrosion remediation techniques on the fascia panels, the panels under investigation were all brought into a similar state of corrosion (as determined by potential and LPR measurements) by means of electrochemical polarisation. The results following remediation for the two corrosion remediation techniques presented in this paper are shown below in Fig. 6 via means of a control chart. The remediation techniques presented in this paper are surface applied Organic Migrating Corrosion Inhibitor (MCI – 2020) and surface applied Silane cream. The results presented in Fig. 6 indicate that each remediation technique alters the corrosion behaviour of the reinforcement in a different manner.

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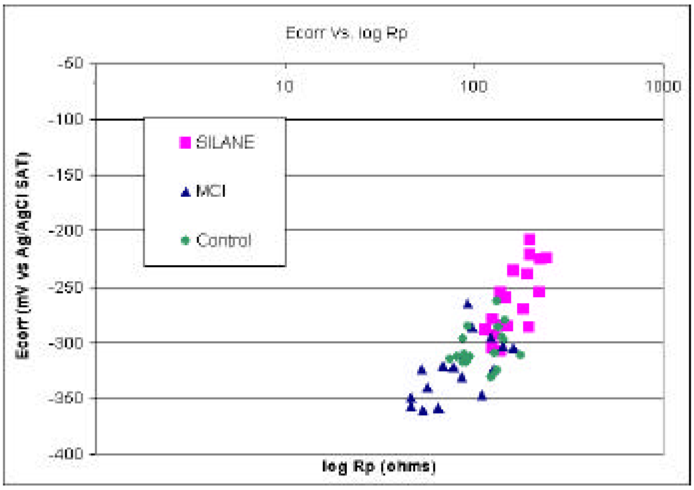


**Figure 6. Variation of log polarisation resistance with half cell potential for remediated fascia panels**

It can be seen that with respect to the control (un-treated) panel, that the MCI actually lowers the half cell potential. Although the MCI tested is a commercial product of which the active ingredients are unknown, this behaviour is common of many inhibitors that are known to be 'cathodic' in nature and may not necessarily indicate that corrosion attack is being intensified. However the decrease in polarisation resistance observed for the MCI treated samples does indicate that at this stage of the treatment (the results reported are up to 6 months following MCI application) when compared to the control panel, corrosion rates are apparently higher for samples treated with MCI. This could indicate that the MCI may be in fact anodic in its action. It should however be noted that such MCI's are dependent for their action on diffusion processes (Phanasgaonkar, 2000) and it may be some time before benefits from such a remediation strategy become apparent.

The panels treated with silane indicate an increase in half cell potential and polarisation resistance when compared to the control panel. The increase in half cell potential with respect to the control may not have been expected if in fact a silane coat can limit the availability of oxygen to the reinforcement, which should ultimately lead to a decrease in half cell potential. However, the polarisation resistance results indicate that corrosion protection is afforded by the silane, although this may not be via limiting oxygen supply to the reinforcement, it may well be due to an increase in the concrete resistivity.

It is apparent that the presentation of results as is done in Fig. 6 presents an attractive and simple procedure for evaluating remediation suitability and effectiveness. Further to this it may also be possible to reveal aspects related to the mechanisms that afford protection in various corrosion remediation measures.

## 6 CONCLUSION

The use of control curves to monitor the corrosion of concrete reinforcement provides a useful way of interpreting data, in that unique criteria can be established for the monitoring of each unique structure. This alleviates a reliance on criteria such as ASTM C876, which have been shown to not be applicable in all cases. Control curves remove the uncertainty that is seen when separately monitoring corrosion potential versus time and polarisation resistance versus time. Furthermore, control curves present a powerful and attractive way of ranking remediation techniques against one another

## 7 ACKNOWLEDGMENTS

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# Overview Of The Service Life And Maintenance Problem Probabilistic Design

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**Summary**: **The service life of a building or structure is determined by the design and construction in combination with the ageing and the maintenance during use. To get grip on this combination it has advantage to present the functioning of the building or the structure on a performance level. As ageing will influence the performance, the combined effect of performance and ageing should be considered. By requiring a minimum limit for the performance the end of the service life or the moment when maintenance is necessary can be defined.**

**It is possible to make a mathematical model for the performances of a building or structure including the process of ageing. It is demonstrated in the paper that this can be done in a similar way as the modelling for the structural design. This is demonstrated by means of a simple beam on two supports that is submitted to a linear degradation process. A calculation example is given in the paper.**

**Once the mathematical model for the performance is available a further step in the modelling is necessary. The parameters in the model will in principle be stochastic; the actual value of a parameter is subjected to scatter. This implies that the probability can only be described by probability density functions for all parameters. Therefore it is necessary to model the uncertainties and add them to the mathematical performance model. By means of an extension of the previous mentioned calculation example it is demonstrated how to do this.**

**The paper finalise with a summary of a probabilistic service life design process that results from the performance and probabilistic modelling.**

**Keywords. Service life, degradation model, probabilistic design, maintenance**

## 1 INTRODUCTION TO AGEING AND MAINTENANCE

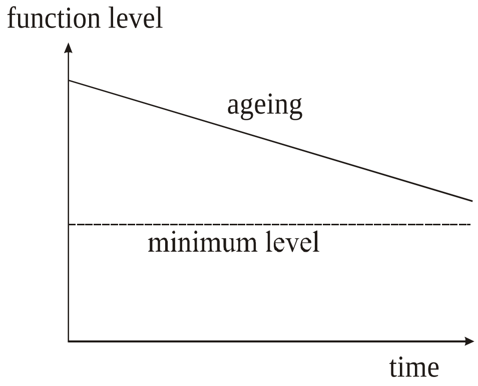
### 1.1 Simplified modelling of ageing and maintenance

During design and construction of a new building or structure during its intended service life his functions fulfil in an adequate way. During the course of time ageing may occur as a result of external or internal factors. To guarantee that all required functions will be fulfilled sufficiently, two in principle different strategies can be chosen:

- The building or structure is over-designed; the protective systems have the capacity to fulfil all functions in a proper way during the whole intended service life (Fig. 1a)
- Intermediate measures are taken to compensate completely or partly the effects of the ageing (Fig. 1b).

The first strategy is called ‘a maintenance free design’. The second strategy is called ‘a design based on maintenance’



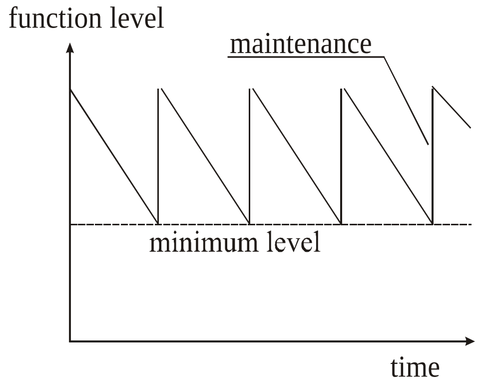




**Figure 1a. Maintenance free** **Figure 1b. Regular maintenance**

Figure 1 gives in principle in a clear way the relationship between ageing and maintenance. In practice it is difficult to find the same clearness. A major reason for that is the scatter in the ageing process and in the effects of maintenance. This same scatter causes finally that the end of the service life has also a big scatter. To give some rough examples:

- Houses 50 – 100 year
- Bridges 25 - 200 year
- Infrastructures 50 - 200 year.

For maintenance measures the effective periods vary also. For example:

- Wall paper 3 - 10 year
- Roofing materials 10 – 25 year
- Pointing and jointing10 – 100 year.

Fine-tuning the service life and maintenance measures is for therefore a complex work. Further we must realise that technical aspects will in many cases not govern the end of the service life of a building or a structure. In reality there are dozens of factors that determine the end of the service life:

- Technical reasons. This is the presented case where ageing determines that the building can no longer satisfy with the functional requirements.
- Economical reflections. It may be for example cheaper to demolish a building and build a new one instead of renovating it. In this case the extra costs should be taken into account, for example the loss of production of a factory or the societal costs caused by traffic yams for renovation a bridge.
- Environmental reasons. An example is a building where asbestos has been used, that can endanger the health of people if it comes into the atmosphere.
- Planning reasons. The demolishing of a building or structures to make room for a new railroad is an example of this.
- Societal requirements. Removing a railroad in a city and replacing it by a tunnel.
- Technological development. Gas factories in cities have been removed and replaced by natural gas pipelines; water towers are replaced by electrical pump to compensate for the pressure fluctuations.

In designing a building or a structure the designer should already in that stage decide in principle on the type of maintenance measures that will have to be taken in the future:

– Strengthening:
  - Higher material quality
  - Better type of material
  - Second option (parallel system)

– Conservation:
  - Applying a protective coating
  - Chemical treatment to make the material durable

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- Sheltering:
  - Covering
  - Hydrophobating
- Preventing the damage
  - Inspection
  - Monitoring.

## 1.2 Typical ageing processes

To make well-funded decision on durability and maintenance we need to have a good knowledge of ageing process of building materials. In other words: the relationships between the level of functioning and the time should be known. These relationships can have different forms. Some characteristic forms are presented in Fig. 2: the ‘linear relationship’, ‘hardening’, ‘weakening’ and ‘spontaneous (sudden) ageing’.

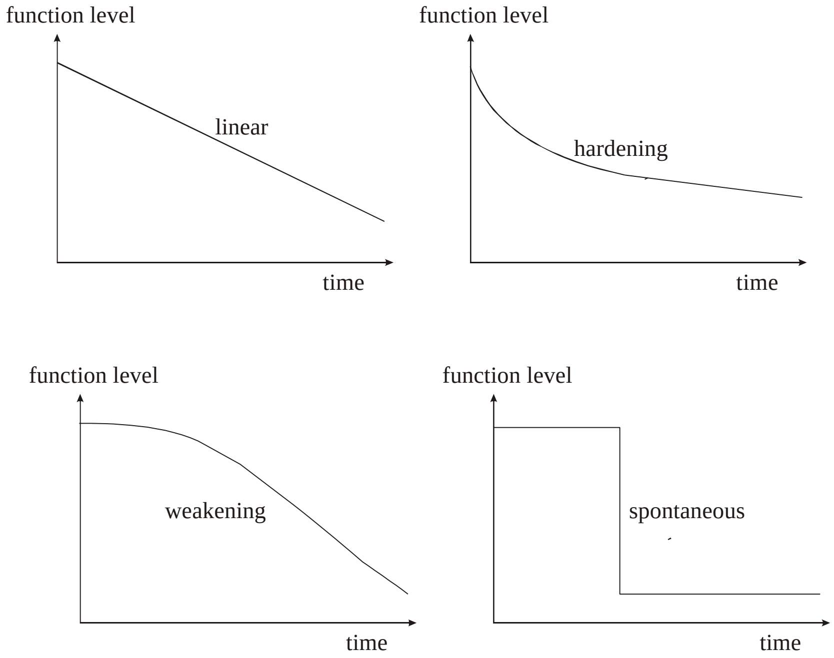


**Figure 2. Typical types of ageing**

The wear of tile or stairs is an example of linear ageing. Strengthening occurs in the case that the inner part of the material has a better quality. This may apply to timber. Weakening is an opposite behaviour. This may also occur to timber in case the better quality is used for the skin and the lesser quality for the interior part. Spontaneous ageing may occur with glass. In the course of time the number of small cracks in the surface is increasing until a level is reached where failure occurs. We also may find this behaviour with fatigue. As number of load or deformation cycles increases the internal damage in the material will increase. At a certain level the balance between the load and the material strength is lost and fatigue fracture occurs. The moment of fracture is in these cases related to the time of exposure.

Calamities lead in general also to spontaneous failure. An example is the breakage of a glass window caused by children playing football. There is no relationship in this case with the exposure period. At each moment of time the probability of failure is the same. In this case we do not use the expression ‘ageing’.

The basis degradation processes, among others, determine the course of the ageing process. We can distinguish between:

- Physical processes, like frost, frost and thaw, wear, transport, settlement of the ground, thermal and hygric deformations
- Chemical processes, like alkali-silica reaction in concrete, corrosion (electro-chemical), acid attack
- Biological processes, like wood rot, algae, acid formation

All these process lead to specific types of loss of function. Some typical ones are shown in the Fig. 2 a to 2 d. The type shown in Fig. 2a can sometimes be seen in the case of acid rain attack on natural stone. Frost-thaw damage of concrete follows Fig. 2b, as the damage penetrates to a limited depth. To increase the depth, we have to wait for a situation with more severe frost.

Settlement of the ground and the damage due to that process follow also Fig. 2b. Figure 2c can be seen in the case of drying of porous media. Figure 2d is related to brittle failure, for example hydrogen embrittlement of prestressing steel, or the disintegration of rubber due to the loss of anti oxidant.

Besides the ageing types of Fig. 2, we often see another type. This is presented in Fig. 3. The ageing process consists of two phases, and sometimes more than two. In the first phase the present protection system looses its function. In the second stage the real actual damaging occurs, if the conditions for damaging are fulfilled. Fig. 3a presents this as a loss of function after the moment that the conditions for damage are fulfilled. The presentation of the increase in damage of Fig. 3b is more often seen in literature. The first stage is called initiation phase and the second one the propagation phase.

a. Loss of function b. Damage

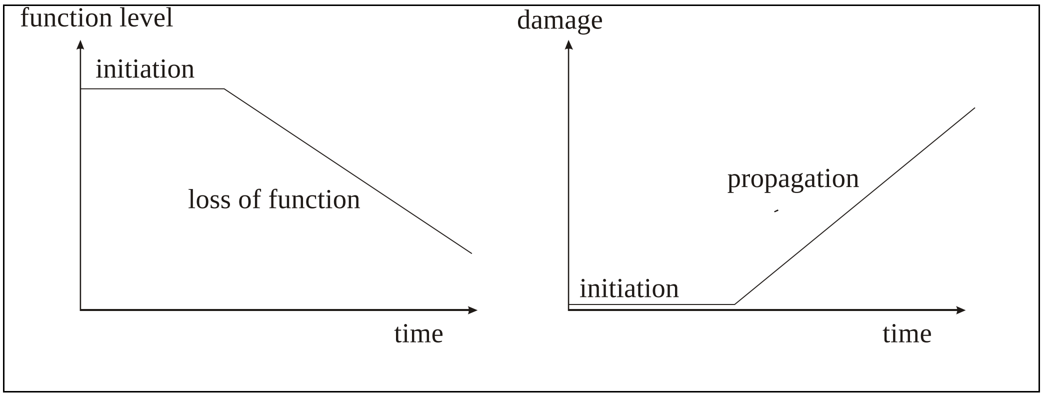


**Fig 3. Initiation and propagation of damage; in fig 3a presented as loss of function and in fig 3b as course of damage.**

An example of ageing, that follows initiation and propagation, is corrosion of reinforcement in concrete. Carbon dioxide or chloride has first to penetrate into the concrete cover in an extent where the protective passivation layer on the reinforcement disappears. Thereupon damage will occur caused by increasing corrosion.

## 2 MATHEMATICAL MODELLING OF AGEING

### 2.1 Why modelling?

General impressions of the ageing of building components are relatively often used to plan maintenance. The service life examples given in the previous paragraph are further general impressions. For many decisions on maintenance this will not be accurately enough and cause subjectivity. The solution for this problem is to increase the amount of relevant information. This can in principle be done in two different ways:

- Collecting data in practice, where the ageing rate is coupled to connected aspects like composition of the building component or structure, and the environmental and use conditions; in principle this is a matter of statistics
- Mathematical modelling of the ageing of the building component or structure; this implies the modelling of the component, the environment, the functioning, and the degradation; this is a matter of stochastics (Siemes *et al* 1984).

The first mentioned way is only possible in the case of fast ageing and the frequent application of the building component. This option is not possible if new materials are used, new environmental conditions apply, or a slow ageing process occurs. The second way is in general preferred, as it gives insight in the ageing processes. It stands open for optimisation and innovation.

### 2.2 Modelling of structural safety and serviceability

The structural design (both safety and serviceability) in modern building codes is based on performances. If the structure functions well, the required performance is present. If this is not true, and the required performance is not there the structure fails. The transition from performing to failure is called a limit state. Structural failures are for example collapse, tilting, overturning, deflection, or vibration. The first three examples belong to the ultimate limit states, and the last two belong to the serviceability limit states.

A mathematical relationship is used to describe a limit state. It comprises of a function for the influence of the load S and a function for the load bearing capacity R. Both S and R may be dependent on several parameters. To prevent that the limit state is exceeded, the following inequality should be satisfied:

$$Z = R - S = R(X_1, X_2, \ldots X_n) - S(X_{n+1}, X_{n+2}, \ldots X_m) \geq 0 \tag{1}$$

Where:

Z - the limit state function

R - a function that describes the load bearing capacity of the structure

S - a function that describes the influence of the load

$X_i$ - a basis variable (i = 1, …., m).

A relative simple calculation example of such a limit state function can be derived from Fig. 4. The beam has two supports and a rectangular cross section. A concentrated load F loads the beam in the middle section. The span is *l*, the width b and the height h.

The maximum bending moments $M_{middle}$ occurs in the middle section:

$$S = M_{middle} = F/2 \;\; l/2 = \tfrac{1}{4} Fl \tag{2}$$

The capacity $M_{max}$ of the middle section is, in the case of linear elastic material behaviour:

$$R = M_{max} = W f = \tfrac{1}{6} bh^2 \; f \tag{3}$$

In which:

W - the section modulus

f - the material strength (in this example either tensile or compressive strength).

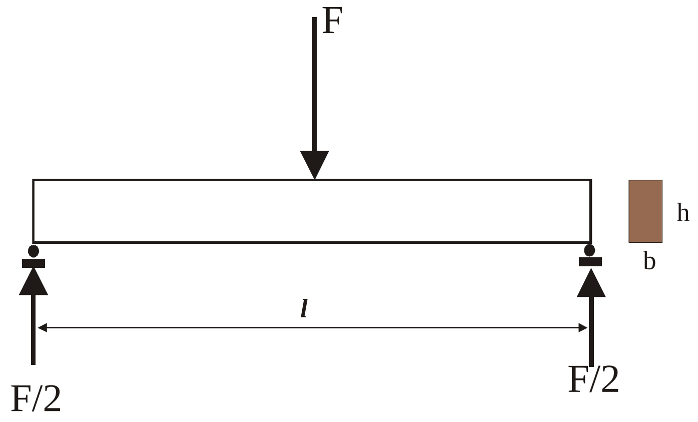


**Figure 4. Beam on two supports**

Equilibrium is possible as long as inequality (3) applies:

$$Z = R - S > 0 \quad \text{or} \quad \tfrac{1}{6} bh^2 \; f - \tfrac{1}{4} Fl > 0 \tag{4}$$

In table 1 the variables are defined. Taking into account the specified values gives:

$$Z = R - S = \tfrac{1}{6}\, 150\; 400^2\; 200 - \tfrac{1}{4}\, 250\; 10^3\; 5000 =$$

$$800\; 10^6 - 312.5\; 10^6 = 487.5\; 10^6 > 0 \tag{5}$$

This means that the inequality is amply satisfied.

**Table 1. Overview of the parameters of the calculation example for the structural safety**

| Variable | Description | Value |
|---|---|---|
| b | beam width | 150 mm |
| h | beam height | 400 mm |
| F | concentrated load | 250 kN |
| *l* | span | 5000 mm |
| f | material strength | 200 N/mm$^2$ |

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## 2.3 Modelling of durability

In the performance based structural design both the resistance R and the load S are considered to be time independent. In many situations this is not realistic. The load can be time dependent or the capacity can change in time due to degradation. Relationship (1) should than be rewritten as a time dependent limit state function (6a, b, c), taking such effects into consideration:

$$Z(t) = R(t) - S(t) > 0 \quad (6a)$$

Or:

$$Z(t) = R(t) - S \quad > 0 \quad (6b)$$

Or:

$$Z(t) = R - S(t) > 0 \quad (6c)$$

In the calculation example given in the paragraph on structural safety and serviceability a simple extension can be made by assuming that due to degradation the width b and the height h of the beam reduce x mm per year (Figure 5). In that case the capacity of the beam reduces according to:

$$R(t) = {}^{1}/_{6}\,(b\text{-}xt)\,(h\text{-}xt)^2\, f \quad (7)$$

The limit state function in this case is:

$$Z(t) = R(t) - S = {}^{1}/_{6}\,(b\text{-}xt)\,(h\text{-}xt)^2\, f \;-\; ¼\, Fl \quad (8)$$

**Table 2. Overview of the parameters of the calculation example for durability**

| Variable | Description | Value |
|---|---|---|
| b | beam width | 150 mm |
| h | beam height | 400 mm |
| F | concentrated load | 250 kN |
| *l* | span | 5000 mm |
| f | material strength | 200 N/mm$^2$ |
| x | degradation rate | 1 mm/year |

In table 2 the deradation rate x = 1 mm/year is added to the variables already given in table 1. With this information we can calculate relationship (8). After t = 50 year the following inequality should apply:

$$Z(t) = R(t) - S = 1/6\,(150 - 50)\,(450 - 50)^2\, 200 - ¼\, 250\, 10^3\, 5000 =$$

$$533.3\, 10^6 - 312.5\, 10^6 = 220.8\, 10^6 > 0 \quad (9)$$

The inequality is still satisfied, and the structure is still safe after a period of use of 50 year.

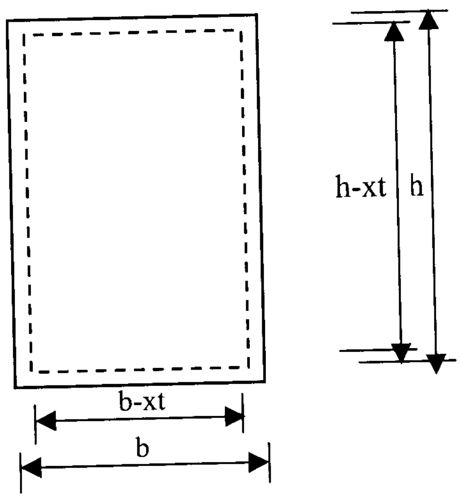


**Figure 5. Calculation example with a degraded cross section**

The modelling in this example is simple. More complex modelling of degradations for concrete structures can for example be found in (Andrade *et al* 1996).

## 2.4 Modelling of uncertainty

We must realise that all parameters in the presented limit state functions are in principle stochastic variables. That means these variables can only be described by means of stochastic functions like the distribution or the probability density function including their parameters mean value $\mu$ and the standard deviation $\sigma$. The standard deviation is a measure for the scatter or the uncertainty of the variable. Three types of sources for uncertainty can be distinguished:

1. Inherent random variability or uncertainty
2. Uncertainty due to inadequate knowledge (model uncertainty)
3. Statistical uncertainty.

The inherent random variability and uncertainty may be split up into two categories: uncertainty that can, and uncertainty that cannot, be affected by human activities. Many kinds of action parameters, for example, wind speed, and earthquake ground motion intensity belong to the second category. The first category concerns, for example, the uncertainties of strength values and dimensions of concrete. These uncertainties can be decreased by the use of more advanced production and quality control methods that, on the other hand, may cause an increase of production costs.

The uncertainties due to inadequate knowledge can also be subdivided into two categories. To one of these categories belong, for example, the uncertainties of physical models or for which knowledge can be increased - and thus uncertainty can be decreased - by research or other similar activities. Also measurement errors belong to this category of uncertainties. To the other category belong, for example, uncertainties that depend on future development. One example may be the future development of the traffic loads on road bridges. The possibility to decrease these uncertainties by research or similar activities is very limited.

The statistical uncertainties are associated with the statistical evaluation of results of tests or observations. The may result from:

- A limited number of test results which cause uncertainties in the estimation of statistical parameters, e.g. mean and standard deviation
- Neglecting systematic variations of the observed variables e.g. of climate variables
- Neglecting possible correlation.
- Increasing test and observational efforts can normally decrease the statistical uncertainties.

We must further realise that the uncertainty can differ with time and place. Together with the different sources for uncertainty, this complicates the definition of the magnitude and the character of the uncertainty.

For the design uncertainty means that it is not enough to calculate the limit state functions like (1) and (6). It is necessary to extend these functions to calculate the probability of failure. For limit state function (1) this means to calculate that the probability of failure $P_f$ is lower than the acceptable failure probability $P_{acc}$:

$$P_f = P\{ R\text{-}S > 0\} = P\{ R(X_1,X_2,\ldots X_n) - S(X_{n+1}, X_{n+2},\ldots X_m) \geq 0 \} < P_{acc} \quad (10)$$

For the limit state function (6) this means to calculate:

$$P_f = P\{R(t) - S > 0\}_T < P_{acc} \quad (11)$$

Note that in relationship (10) and (11) the inequality sign has changed as we now calculate the failure of the structure and no longer the performance.

To calculate the failure probability we have the following options for making the necessary probabilistic calculations:

- To use statistics; as we already have mentioned this will only be possible if much relevant data is available
- To use stochastics; this will be the main option.

To make fully correct probabilistic calculations is in practice often not a realistic option. The main reasons for that are that the limit state functions are often not linear and the probability density functions of the parameters are in general complicated and will differ from normal distributions. This means that we often have to make simplifications. For structural calculations the following levels of simplifications have been agreed by the JCCS (Joint Committee on Structural Safety):

Level 1: deterministic calculations with safety margins that take into account the effects of the stochastic character of the variables.

Level 2: probabilistic calculations with agreed simplifications.

Level 3: fully correct probabilistic calculations.

This paper will deal with the level 2 and level 3 approaches. It is however possible to make simplified service life calculations. Further simplifications are possible to use the so-called factorial method from report [y] is related to the level 1 approach, bur formally it is not a level 1 approach as the (safety) factors that are used for calculating the service life are not derived on basis of level 2 or 3 calculations. At this moment these factors are based on ‘good engineering judgement’.

In practice the level 2 approach is the most commonly used probabilistic level. Known approximations are for example the Monte Carlo simulation and the FORM (First Order Reliability Method). The latter is based on linearisation of the limit state function and replacing non-normal distributions by equivalent normal distributions. The term ‘reliability method’ refers to the fact that in most cases the reliability index $\beta$ is calculated instead of the failure probability $P_f$. There is however a direct relationship between these two parameters:

$$P_f = \theta(-\beta)$$

Where:

$\theta$ - function of the standard normal distribution (mean value $\mu = 0$ and standard deviation $\sigma = 1$)

The main reasons to use the reliability index instead of the failure probability are:

- Presenting the failure probability gives the impression of an exact calculation result
- The reliability index is directly related to the performance of the structure
- The reliability index is a direct result of the FORM calculation
- The notation of the reliability index is simpler than the notation of a failure probability.

If we calculate the reliability index for the first calculation example in the paragraph on modelling of structural safety and serviceability, we first have to define the probability density functions for the parameters in the limit state function. As an example this has been done in Table 3. The parameters are similar to the paramaters given in the calculation example on structural safety in Table 1. For the parameters with a deterministic distribution it has been assumed that the scatter of these variables is neglectable. The distributions for F, *l* and f are just chosen for demonstration.

**Table 3. Overview of the stochastic parameters of the calculation example on structural safety**

| Variable | Description | Distribution | Mean | Standard deviation |
|---|---|---|---|---|
| b | beam width | deterministic | 150 mm | - |
| h | beam height | deterministic | 400 mm | - |
| F | concentrated load | normal | 250 kN | 75 kN |
| *l* | span | deterministic | 5000 mm | - |
| f | material strength | normal | 200 N/mm$^2$ | 20 N/mm$^2$ |

On basis of this information reliability index $\beta = 4.0$ has been calculated by means of a FORM approach (First Order Reliability Method). The Eurocode 1 requires for an ultimate limit state a reliability index of at least $\beta = 3.8$. The conclusion is therefore that this design is adequately safe.

A further step in the calculation is to extend the calculation to the example given in the paragraph on modelling of durabilty. In Table 4 the stochastic parameters are defined including the parameters for the degradation rate x. The basis of these parameters is similar to the parameters already specified in Table 2 for the calculation example on the modelling of durability.

**Table 4. Overview of the stochastic parameters of the calculation example**

| Variable | Description | Distribution | Mean | Standard deviation |
|---|---|---|---|---|
| b | Beam width | deterministic | 150 mm | - |
| h | Beam height | deterministic | 400 mm | - |
| F | concentrated load | normal | 250 kN | 75 kN |
| *l* | span | deterministic | 5000 mm | - |
| f | material strength | normal | 200 N/mm$^2$ | 20 N/mm$^2$ |
| x | degradation rate | lognormal | 1 mm/year | 0.05 mm/year |

The results of the FORM-calculations are presented in Fig. 6. It is obvious that in course of time the reliability index $\beta$ will decrease. At t = 0 the reliability index starts with the previous value $\beta = 4.0$. After about 4 year the reliability index has reaches a value of 3.8, being the minimum requirement according to the Eurocode. The idea to use the same format for structural design and durability desing has been given by Siemes and Rostam in 1996 for the service life deisgn of concrete structures.

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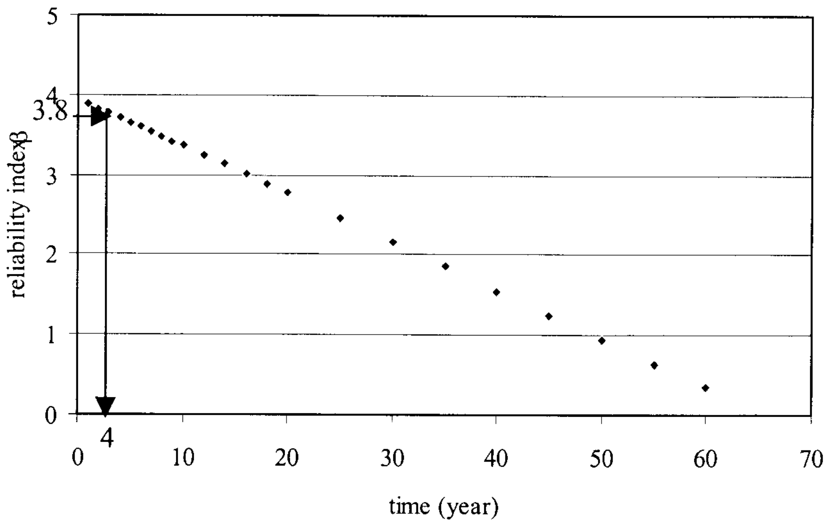


**Figure 6. Results of the probabilistic calculation of the degrading beam**

## 2.5 Summary of the probabilistic design process

In case of ageing the designer should take the following actions to make a probabilistic calculation:

1. Define the functions that have to be fulfilled. In general this will be a set of functions that may differ for the various components of the building. The number of combinations of functions and components may be come very high. The designer should for that reason identify, already in the earliest stages, what are the most critical combinations, and give priority to them.
2. Establish a limit state function related to all critical combinations of functions and components.
3. Find out what are the relevant types of ageing that can influence the functioning of the building. The type and the intensity of the ageing are dependent of the design and the construction and further of the use and external factors like the environment. This is another reason to identify the most critical situations.
4. Extend the limit state functions with the effects of ageing.
5. Define for all parameters in the limit state functions (including ageing) the probability density, the mean value and the standard deviation.
6. Make a probabilistic calculation.
7. Define the required reliability (i.e. the probability that the building will not fail to fulfil its functions). The Eurocode gives for structures already some values. For non-structural buildings the designer should take a decision after consulting the new owner. In general these decision must be taken on basis of the risk of non-functioning. In this respect risk is defined as the multiplication of the probability of failing to fulfil the functions and the amount of damage that will be involved in such a failure. To manage the risk the failure probability should be lower as the failure costs are higher. In many decisions we already act in this way, but in an implicit way. However as a designer we should do it more explicit

The design examples given are based on direct degradation of the material of the structure. This does not reflect all options that the designer has available. Different types of limit state function will be needed in case the following additional measures are provided:

- Strengthening:
  - Second option (parallel system)
- Conservation:
  - Applying a protective coating
- Sheltering:
  - Covering

- Hydrophobating
- Preventing the damage
  - Inspection
  - Monitoring.

The failure function modifies to:

$$P_f\{\text{total system}\} = P_f\{\text{additional measures}\} \cap P_f\{\text{original structure}\}$$

Calculating the combined failure probabilities will complicate the design, especially if the two separate failure probabilities are dependent. A real complication occurs in case a parallel system like a cathodic protection is applied or in case the required reliability depends on inspection or monitoring. The presented limit state approach will in general not apply and a separate reliability analysis will be needed.

The presented framework in this chapter is based on service life design methods as they are used for concrete structures (DuraCrete, 1999). A practical application of this design method is the service life design of the Western Scheldt Tunnel in the Netherlands. This bored tunnel is the first structure that was designed with the DuraCrete service life design method (Breitenbüchner *et al* 1999). The framework is in principle material and component generic. No formal restrictions were found in doing so. This is in line with earlier publications on this subject (Siemes and Edvardsen, 1999).

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# Overview Of The Development Of Service Life Design For Concrete Structures

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**Summary**: **After the introduction of reinforced concrete it was believed that the material was extremely durable. Soon it was found that reinforced concrete could have serious durability problems and that special care should be taken to avoid them. Durability became a design issue.**

**Based on experience from practice and research, construction rules have been formulated to ensure the durability of the reinforced concrete. Durability was twenty-five years ago however no real issue for design and practice. National organisations and international organisations like CEB and RILEM have become since than very active and widened the fundamental and practical knowledge on durability. This forms the basis of the present design manuals, standards and codes on durability of concrete.**

**The present durability method is based on a vague idea about the service life of the structure, being some decades. In a number of cases is however extensive maintenance and repair necessary. In a few cases lack of durability has even caused the collapse of concrete structures. This has initiated new research to the various degradation mechanisms. It changed also the approach to the problem: the service life has been taken into account explicitly. The design changed from a deem-to-satisfy approach to a performance-based approach with explicit attention for the design life, limit states and reliability. CEB has decided in 1996 to accept this approach as the basis for durability. In the mean time the research project DuraCrete (in the 4th framework programme Brite EuRam of the European Union) has been started to produce a first manual for design and assessment on this basis. This became available in the course of 1999. The knowledge of this project has been used recently to make the service life design of the whole Western Scheldt tunnel (bored tunnel in The Netherlands). This is the first time that a complete concrete structure has been designed on basis of service life, performance and reliability.**

**The history of service life design of concrete structures will not be ended by these achievements. Further knowledge, design methods, new materials, construction techniques, and so on need to be developed for the further improvement of concrete structures.**

**Keywords. concrete, durability, service life design, performance**

## 1 INTRODUCTION

### 1.1 History of durability in concrete

When the gardener Monier combined concrete with a wire mesh he intended to improve the tensile properties of the concrete. Probably unintended he realised in that way a very durable structural material: reinforced concrete. In the course of time some durability problems occurred. The first durability requirements appeared in the concrete codes that were published after the Second World War. In the meantime we have reached a situation where the codes are based on deem-to-satisfy rules. These are based on a combination of experience, research, and intuition (good engineering judgement). For most of the environments and concrete structures this approach has the advantage of being simple, experienced, and reliable.

This conventional approach has however disadvantages:

- it can not be used for structures with a service life that differs from the usual service life (about 50 year); this applies especially for structures with an intended long service life like infrastructures
- it can not be used for new materials, such as the new cement types that are developed nowadays
- it can not be used for new types of structures or new environments.

In general it can be stated that the conventional approach cannot be used under circumstances where we have a lack of experience. Through the years we have therefore seen a growing need to make designs for concrete structures based on a distinct, relatively long, service life. The development of durability requirements can be shown on basis of the design of real concrete structures in the past decade. This will be shown on the basis of some special concrete structures, like storm surge barriers and a bored tunnel that have been built in The Netherlands.

### 1.2 Noordersluis in IJmuiden

This is the biggest water lock in The Netherlands and has been built in the end of the twenties. The lock provides the connection between the North Sea and Amsterdam. In the twenties this water lock did not have the explicit function of a storm surge barrier. Nowadays it is considered as such. In that period durability was no issue. The construction of this lock has been used to improve the practical knowledge on concrete technology. For the construction of the lock many different types of cement (various types of Portland and blast furnace slag cement) and many concrete compositions have been used. The testing results of the concrete properties (both technology and strength) have been presented in small reports that were available to all interested people. Durability was in that period no explicit item for the design and construction of this water lock. At this moment reconstruction activities are carried out, mainly to repair damages that were the result of sabotage by the occupiers during the Second World War. Further shortcomings are the presence of alkali-silicareaction in parts of the structure, leaching, and a low tensile strength.

### 1.3 Haringvlietsluizen south of Rotterdam

These sluices play an important role in the control of the water outlet of the rivers Rhine and Meuse in the Netherlands. The sluices were one of the first Delta works that have been built in The Netherlands after a storm surge in 1953 has caused the flooding of a south–western part of the country. Due to this flood about 1800 people were killed. During expected storm surges the sluices can be closed.

The Haringvlietsluizen (Fig. 1) have been built in the sixties without any specific service life requirements. The design philosophy was simple: make a durable and robust structure. Based on the existing knowledge the following main measures have been taken:

- the use of blast furnace slag cement
- water/cement ratio lower than 0.45
- concrete cover of at least 70 mm
- prestressing where possible

The sluices are at present about 35 years old and show no serious durability problems. A small durability problem is present near the centring pins where a low quality mortar has been used to fill the holes.

**Figure 1. Haringvlietsluizen**

### 1.4 Storm surge barrier in the Eastern Scheldt

This barrier (Fig. 2) has been built in the eighties. Like the Haringvlietsluizen it is an open barrier that only will be closed if a storm surge is expected. With a total budget of about 3,5 billion Euro this is the most expensive structure ever built in The Netherlands. Because of this high amount of money and the growing awareness of the safety aspects of such a storm surge barrier the durability requirements were extreme. The design service life was set on 200 year! To achieve this long service life similar measures have been taken as for the Haringvlietsluizen.

**Figure 2. Storm surge barrier in the Eastern Scheldt**

Design calculations that have been made, showed that the concrete cover would protect the reinforcement against corrosion for at least 80 year. In cracked areas the protection period was estimated as at least 30 year. These restricted periods have been excepted as it was found that improving the protection period would cost more money than the replacement of parts of the concrete cover after 30 year respectively 80 year.

### 1.5 Storm surge barrier in the Nieuwe Waterweg (north of Rotterdam)

This is the last structure that has been built in the framework of the Delta plan (Fig. 3). It has been built in the beginning of the nineties. The design service life has been set on 100 year. To achieve this service life the following consideration has been made. The requirement for the concrete cover in code of that moment was at least 35 mm in combination with a water/cement ratio not higher than 0.50. The general impression at that moment was that these requirements would lead to a concrete structure with a service life of at least 50 year. To extend this service life to 100 year the value of the concrete cover has been increased with a factor $\sqrt{(100/50)} = \sqrt{2}$, giving a round value of 50 mm. The square root function has been applied as the ingress of chloride into the concrete roughly follows this function.

**Figure 3. Storm Surge Barries Nieuwe Waterweg**

### 1.6 Bored tunnel in the Western Scheldt

Building bored tunnels in soft clay and sand is a relatively new development in the Netherlands. First a pilot tunnel was built and now the construction of a 6.5 km long tunnel with two tubes under the Western Scheldt is in progress (Breitenbücher et al, 1999). The contractor had to prove that this tunnel has a service life of at least 100 year. No method has been prescribed to prove this service life. Further it has not been defined for which part and for what functions and performances of the tunnel this service life had to prove.

The investor and the contractor decided finally that the DuraCrete methodology, the knowledge, and data from this project should be used as a basis for the service life design. The requirements with respect to the reliability indexes for the ultimate limit states and the serviceability limit states have been copied from the Dutch Building Decree. The limit states with respect to prevention of corrosion have been considered as serviceability limit states. This is a simplified approach, as these are in principle economic limit states. For economic limit states the reliability index should be defined on basis of an economic optimisation.

The service life design of the Western Scheldt Tunnel, both the prefabricated bored tunnel (Fig. 4) and the entrances, is the first concrete structure that has been designed on basis of a probabilistic approach and on basis of performances. Even some special parts made of steel have been designed on this basis.

**Figure 4. Lining of the Western Scheldt Tunnel with prefaricated concrete segments**

### 1.7 Cargo railway link between Rotterdam and Germany and the High Speed Railway Link between Amsterdam and Brussels

The cargo railway link and the High Speed Railway Link are intended to improve the infrastructure in the Netherlands. The construction costs for each of both projects are estimated on 4 billion Euro. All structures will be made of concrete. The durability requirement is that these structures should have a service life of at least 100 year.

### 1.8 Summary

The presented examples show clearly that owners and investors of important infrastructures want to have more durability for their structures than for more common structures. This was even true some decades ago. In practice the higher durability was achieved by paying more attention to the construction works and by making the durability requirements in codes and standards more strict. Although some attempts with service life calculations were made, this failed as no proper methods were available and a design framework was lacking.

## 2 REQUIREMENTS FROM THE OWNER

From these examples it can be concluded that owners of structures realised the need for a long service life for important and expensive structures. This is in line with the demands from the society. The methods used to prove the service life were crude and based on a deterministic view. The scatter in the service life has been totally neglected. Looking back it can also be stated that the service life design was mainly the job of structural engineers. Material scientists and engineers played no import role in the durability design. This situation was also present in many other countries in that period.

For structures like the Great Belt Link in Denmark and the Oresund Link between Denmark and Sweden material specialists have used improved degradation models for the service life design. These models were however still be used as mainly deterministic models.

The present design approach with respect to durability of concrete structures is based on a reasonable understanding of the main degradation processes for concrete, reinforcement and prestressing steel. The performance of the design is however not explicitly formulated as a service life. It is based on deem-to-satisfy rules (for example minimum cover, maximum water/binder ratio, and crack width limitation) and the assumption that if these rules are met, the structure will achieve an acceptably long but unspecified life. The information about the service life to be achieved is to a large extent empirical. Improving the durability increases building costs without any quantification of the reduction of maintenance costs or failure costs. Current design methods only permit to calculate the whole life cycle costs from assumptions with respect to maintenance and failure rates. There are thus no objective means for demonstrating that future maintenance and repair costs will be acceptably low.

This common design approach to durability has other disadvantages. The rules are inadequate in some aggressive environments, while they are too rigorous in other environments. In some cases, this results in a 'belts and braces' approach (many different types of measures on top of each other), which may contain unnecessary and even counteractive measures. Lack of durability can cause serious safety and serviceability problems for structures. Despite this, designers have usually considerably more attention for load and resistance based structural design than for durability design. Recent history has however shown that due to a lack of durability, serious collapses and other types of damages may occur with large amounts of damage, maintenance and repair.

To improve this situation a new concept for durability design needs to be established. Similar to the current procedures for structural design, a design for durability should be performance based taking into account the probabilistic nature of the environmental aggressiveness, the degradation processes and the material properties involved.

In order to quantify the durability the concept of a service life design has been introduced. In this respect the performance requirements for a service life design as stated in the CEB-FIP Model Code 1990 has been adopted: *'Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time without requiring unforeseen high costs for maintenance and repair.'*

Such a rational design for durability, however, requires both an overall methodology and calculation models for the actual degradation processes of concrete structures. Similar to the structural design code for loads, safety requirements and limit states must be defined for the design service life.

## 3 DEVELOPMENT OF SERVICE LIFE DESIGN KNOWLEDGE

### 3.1 Rilem

Within the international organization Rilem (International Union of Testing and Research Laboratories for Materials and Structures) many technical committees have contributed to the understanding of degradation mechanisms for concrete, reinforcement and prestressing steel. The accompanying tests were developed and evaluated in round robin tests. Despite this effort it was not possible yet to understand all relevant degradation mechanisms. With respect to, for example, frost attack, the influence of frost and de-icing salts, alkali-silica reaction, or alkali-carbonate reaction, the essential knowledge to fundamentally understand the mechanisms is still lacking.

The main strategy in the durability work of Rilem was to avoid these adverse reactions. Figure 5 presents a scheme with all adverse effects that can occur due to corrosion. The following two main stages can be distinguished:

- initiation of the corrosion process due to carbonation of the concrete or ingress of chloride
- propagation of corrosion of the reinforcement

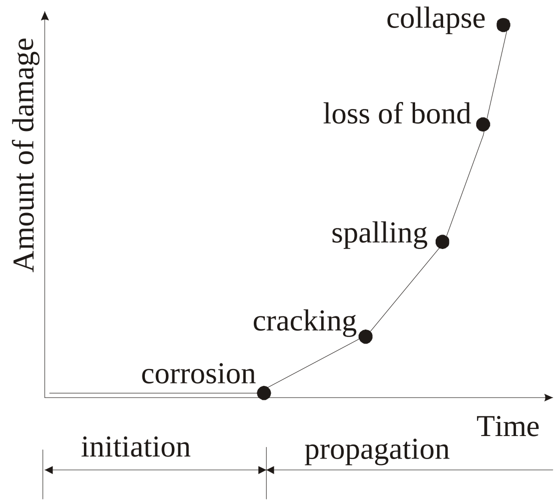


**Figure 5. Various adverse events during corrosion of the reinforcement**

In the propagation phase several adverse events can happen:

- cracking of the concrete cover
- spalling of the concrete cover (parts of the cover come loose)
- loss of bond of the reinforcing bars
- loss of cross section of the reinforcing bars
- loss of bearing capacity of the structure, resulting in collapse.

For corrosion of steel in concrete this means that much attention has been paid to initiation processes like carbonation and ingress of chlorides. These processes consume much time before a situation is achieved where corrosion can start. Little is however known about the corrosion process itself. There we must think about de dependency on fluctuating temperature and humidity, cracking and scaling of the concrete cover, loss of bond and finally collapse.

The strategy ‘avoiding reaction’ instead of ‘defending against reaction’ was also followed for other types of degradation reactions. For example for frost, frost in combination with the presence of de-icing salts, acid attack, and sulphate attack. To prevent the both frost mechanisms deem-to-satisfy rules had to be followed with respect to the maximum value of the water/cement ratio, or to the amount of entraining of small air bubbles in the concrete. For acid or sulphate attack rules with respect the composition of the cement had to be followed.

In the building industry the interest has grown during the last years towards a more performance based approach. The same applies to the work on durability of concrete from Rilem. Examples are the report on performance criteria for durability of concrete from Kropp & Hilsdorf, 1995 and the report on probabilistic service life modelling for concrete structures edited by Sarja and Vesikiari 1996. Publications like these opened the way to service life design on the same basis as other design items like structural integrity.

### 3.2 Comité Euro-International du Béton (CEB)

The ‘Comité Euro-International du Béton’ (CEB) has played an important role in providing a platform for discussion on design of concrete structures. The results of this discussion were published in reports that were named ‘bulletins’. These bulletins were u often used as a basis of design or as a basis for national or international codes. CEB published her first bulletin in 1957, followed by a long row of reports on mainly structural aspects of concrete. Twenty-five years later in CEB Bulletin 148, 1982, a state-of-the-at report on durability was published. This was followed in 1984 with a report of an international workshop. In 1987 the first CEB guide on durable concrete structures followed. The present concrete codes in Europe are mainly based on these bulletins.

A new approach to durability design was published in 1997 in CEB Bulletin 238. By means of an example for carbonation induced corrosion, a performance and probability based service life design was introduced. The new approach combined the existing probabilistic framework for structural design with service life models that described the degradation processes in concrete. On this basis it was in principle possible to choose between different design options, for example increasing the concrete cover or decreasing the water/cement ratio. According to the conventional codes there is a minimum requirement for the cover and a maximum for the water/cement ratio. These requirements must both be fulfilled.

### 3.3 DuraCrete

The CEB Bulletin 238 presented only the principle of the new approach and gave one example. It will be clear that this is insufficient for a design guide. That should contain at least the following items:

- the framework for the design process
- definition for the design service life (reference period)
- definition of the required performances
- definition of the required reliability, related to the performances and the design service life
- models for the loads, the aggressiveness of the environment, the structural response, and the degradation of the materials.

The amount of work to be done was too huge to perform within CEB. Twelve organizations in Europe initiated therefore the research project DuraCrete. That project was co-financed by the European Union. It took 3 years to perform the project, that was organised in separate tasks (Fig. 6).

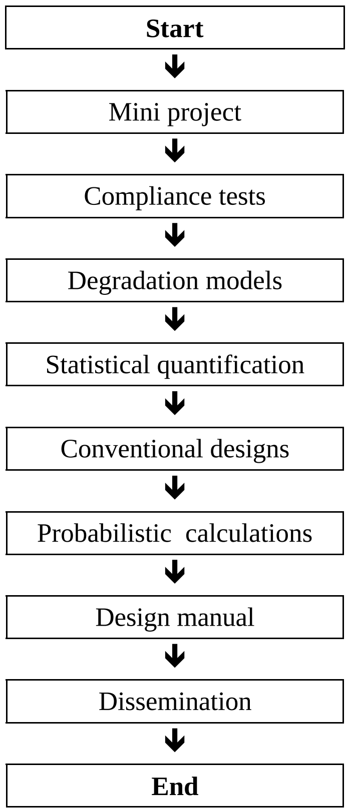


**Figure 6. Flow chart of the DuraCrete project**

The project was mainly based on existing data and knowledge on durability. The work focussed on

- collecting degradation models and environmental models, and find consensus about them; with respect to structural models and loads the Eurocode and the CEB Model Code were used
- developing compliance tests to define the durability characteristics with respect to concrete
- statistical quantification of the parameters in the models
- designing a number of structures according to the various national concrete codes in Europe
- making probabilistic calculations of the previous designs to find the reliability levels that are present in these codes; this result is used as a basis in the new design manual
- preparing a manual for probabilistic and performance based service life (re)design.

The DuraCrete project ended with a guide for the design of new concrete structures and the redesign of existing structures (DuraCrete, 1999). The background of the method and the knowledge and data that was gathered was published in some fifteen reports (DuraCrete, 1999-2001). Besides these results it was important that the work of material scientists, concrete technologists, structural engineers and designers, and reliability engineers was integrated. Further it was important that we are now able to replace the passive durability approach in the present codes by an active service life design. The project demonstrated that it is even possible to make a simplified service life design on basis of safety margins and characteristic values instead of the probabilistic design.

It is obvious that DuraCrete has not resulted in a final design manual. Many new things have to be developed. Missing degradations like alkali-silica reaction, and frost attack have to be added. The design models must further be calibrated to the behaviour of existing structures. Finally it is necessary to make the modelling more unambiguous. In the present form only experts can use it safely, because they can identify ‘bugs’ and they are able to choose the proper models in case of non-standard situations. These are also conditions for standardisation of the method.

**Darts**

In the DuraCrete project a perfect match was made between structural design and durability design. This makes it possible to optimise with respect to both aspects. For many designs this is however not enough. There are more aspects to consider. In a new research project ‘DARTS’ (Durable And Reliable Tunnel Structures), that is co-funded by the European Union an integrated design method will be developed in the period from 2001-2003. The method will be restricted to the design of tunnels. The restriction was made to reduce the amount of work. Tunnels were chosen because of their high costs and social importance. The method can be extended with other types of structures in a later stage. The design aspects that will be considered are structural behaviour, durability, hazards, sustainability, and socio-economic aspects. A flow diagram is given in Fig. 6.

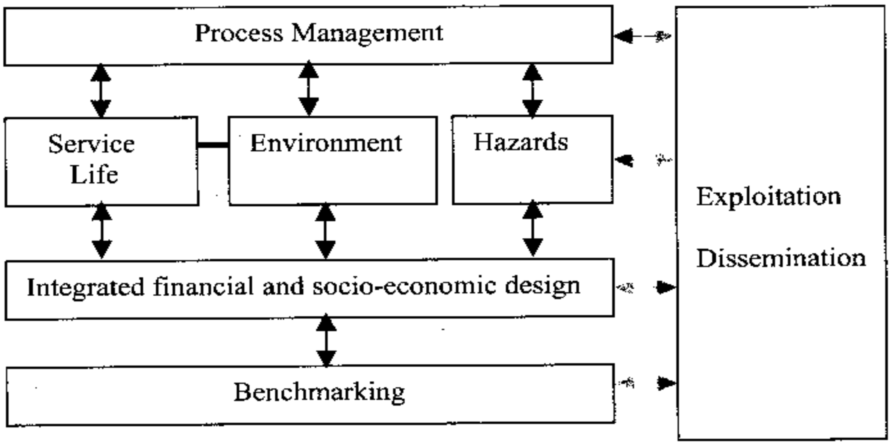


**Figure 6. Flow chart of the Darts project**

The project Darts is connected to various research related networks that are active in the field of tunnel fires and safety.

### 3.4 Comité International du Bâtiment (CIB)

The activities of the Comité International de Bâtiment (CIB) are not specific directed to concrete applications. Some of the activities with respect to service life design are however material generic and are therefore also relevant to concrete. One of the scientific committees working on service life design is a joint committee between CIB and Rilem: CIB W80 / RILEM 175 PSL ‘Prediction of Service Life’. The present work of this committee relates to service life models. Recently a task group has been established to develop performance-base methods of service life design based on models of degradation and environmental actions in three different, but coherent levels:

- a fundamental and scientific approach and provide framework for different levels of design
- a simplification of scientific models to engineering design
- a development of simplified and practical design approach (factorial method).

The line of approach of this joint committee is parallel to the recent developments of service life design for concrete structures. This could mean that in the near future the concrete approach can be embedded in a generic performance and probability based framework for service life design.

## 4 RECENT APLICATIONS IN THE NETHERLANDS

The construction activities in the Netherlands have increased in the past period with the objective to improve the infrastructure. These activities will prolonged for at least the next decade. Some of the special projects are:

- Second Heinenoord Tunnel; this project has been finished; it is the first bored tunnel that was built in the Netherlands; the dimensions of the tunnel are relatively small (two tubes with a length of 950 m and outer diameter 8.30 m), because the construction served mainly to study the special items related to boring tunnels in soft soil
- Western Scheldt Tunnel; this is a two-tube bored road tunnel, with length of 6500 m and an outer diameter of 11 m; this is the first concrete structure that has been designed for 100 year service life on basis of the method developed by DuraCrete; the budget of this project is about halve billion Euro
- High Speed Railway Link Between Amsterdam and Brussels In Belgium; in this link various structures like bridges and tunnels are involved; one important tunnel is the Green Heart Tunnel; this is a single-tube bored tunnel with in the

middle a separation wall and a length of 7 km and a diameter of 14.9 m (world largest diameter for a bored tunnel); the budget is for the civil structures in the link is about 4 billion Euro

- Cargo Railway Link Between Rotterdam and the Ruhr Region in Germany; the length of the link will be 160 km; it includes about 160 civil engineering structures including a seven kilometre long bored tunnel; the budget is about 4 billion Euro.

Giving the great importance of these project they have in common that the owners required a design service life of at least 100 year. Despite of this requirement no special performance based service life design was made for the Second Heinenoord Tunnel. After construction it could however be demonstrated on bases of the DuraCrete models that the expected service life will be substantially lower (Siemes & De Vries, 1999). The low service life was the consequence of a series of weaknesses in the design such as a low concrete cover, ignoring the presence of de-icing salt, and a wrong cement type.

The other mentioned structures are all designed for at least 100 year. For the design and construct contract of the Western Scheldt Tunnel (Breitenbüchner *et* al, 1999) the contractors made probabilistic, performance based designs. For the Green Heart Tunnel these probilistic calulation were required in the design and construct contract. For the other structures the design calculations were made by the owner. The results of the design calculations were translated to requirements with respect to a combination of the concrete cover, the cement (binder) type and the diffusion coefficients for chloride and carbonation.

## 5 CONCLUSIONS

Reinforced concrete structures are used in practice for more than a century. Only after the Second World War special attention was paid to the durability of the structure. In the course of time this resulted in concrete codes with deem-to-satisfy rules (recipe's) to guarantee the durability. These rules were based on a mix of experience and research. International research organizations like Rilem and CEB provided the necessary consensus, encouraged further research, and developed guidelines and model codes.

Owners who intended to built special structures, such as big infrastructures, were not always satisfied with the durability level in the existing concrete codes. Therefore they wished to have structures with an extra long service life. In general they required at least 100 year and sometimes even 200 year. It was unclear what they really wanted to achieve, as they did not define the related performances and the design method to be used. Nevertheless special provisions were taken, that in general improved the durability.

In the recent period the attention in the construction industry changed to performance and probability based design. This was developed for structural design to a high degree and implemented in the codes. On this bases new service life design methods were developed. They combined the concrete structural design methodology with environmental loads and degradation models. This finally resulted in the design methodology DuraCrete.

Various practical projects that recently were designed according to the DuraCrete method proved that it can be used for the service life design of concrete structures. The method is however only reliable if applied by experienced material scientists, with good capabilities to overview the consequences of the chosen models. To get a broad use it will be necessary to give more guidance to this aspect. Further some bugs should be repaired, and more relevant degradation mechanisms should be added.

It is expected that this service life design method for concrete structures will be further developed in the near future and will be made suitable for standardisation. A similar development can be expected for other structural materials. For example the progress of service life design of timber structures is in this respect promising (Foliente *et al* 1999).

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# Predictive Models Of Deterioration Rates Of Concrete Bridges Using The Factor Method Based On Historic Inspection Data

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**Summary: Recent decades have seen rapid developments in structural research attempting to predict the service life of building materials and components. Earlier ideas, ambitions or visions among a few researchers, are today realities or within reach for engineering applications. The reasons for these developments include improvements in testing procedures, better analytical tools and methods, and a computerisation that has significantly facilitated the ability to analyse and process large data sets. Another important influence and driving force for continued research in this area has been the demand for reliable service life data from building asset/property managers, management consultants and building owners. The risk of deterioration of reinforced concrete structures has highlighted the importance of developing service life prediction models so that optimal strategies for their maintenance and repair can be developed. Many approaches have been suggested to estimate the service life of concrete structures. To date, they are usually based on the Factor Method, or a stochastic/probabilistic approach. In this paper, data on bridge deterioration of over 400 bridges in the UK will be used to provide predictive models based on the factor method.**

**Keywords. Concrete Bridges, Rates of Deterioration, Service Life, Modelling, Factor Method.**

## 1 INTRODUCTION

Reinforced concrete provides a relatively inexpensive and durable material that has become widely used in construction of roadways and bridges. Corrosion damage has become a major threat to bridge durability in the UK as in all parts of the world. The inherent risk of deterioration of reinforced concrete structures due to corrosion has highlighted the importance of developing service life prediction models so that optimal strategies for their maintenance and repair can be developed. According to Clifton (Clifton, J. R., 1993), the methods applicable to estimate the service life of concrete subjected to deterioration processes are based on: (a) experience; (b) performance of similar materials; (c) accelerated testing; (d) mathematical models that can describe the chemistry and physics of the degradation processes; and (e) the application of reliability and stochastic concepts. Different approaches have recently been developed to estimate the service life of concrete structures, to date they are based on the concept of delay time, the Factor Method, or a stochastic/ probabilistic approach. These models rely on sound engineering judgement or statistical information in order to determine the structure's deterioration rate. A key factor missing is that no 'actual data', collated over the past number of decades, has been used in their development. An important influence and driving force for continued research in this area has been the demand for reliable service life data from building and other structures asset/property managers, management consultants and structures owners.

This paper reports the modelling of the deterioration of concrete bridges using a large database of concrete bridge inspection records (Christer, A. H. et al, 1993), (Mc Parland, C. B. et al, 2001), (Redmond, D. F. et al, 1997), (Rigden, S. R. et al, 1993), (Rigden, S. R. et al, 1996). The defect histories of 725 concrete members from 439 bridges in the UK were obtained from inspection record covering a period of over 50 years. Of the information recorded and stored in the database, perhaps the most relevant data was the component age and exposure conditions. Through their service life, the various components of a structure will undergo different stages of deterioration, starting with cracking, minor spalling, and extensive spalling which might lead to total failure. It is evident that if enough data were available it would be possible to establish the age at which a structural component will have deteriorated to a specific phase of its service life. This paper reports on a simple spreadsheet program, based on the principles of the Factor Method. The data collected from the inspection records is used to calculate the relevant factors associated with the Factor Method. Subsequently, a critical appraisal of the spreadsheet is offered in order to evaluate its suitability in the field of service life prediction.

## 2 FACTOR METHOD

Research into the service lives of structures has been ongoing for over two decades. The need for qualified predictions of the service life of structures and their members has increased. Deterministic and probabilistic approaches for such predictions were developed. However, up until the last decade, a sufficiently general and simple method had not been developed. Efforts towards this resulted in 1993 in the creation of the Factor Method. This approach is based on a concept developed by the Architectural Institute of Japan (AIJ, 1993) and Construction Audit Ltd (CAL, 1992). It is used as a method for estimating the expected service life when more detailed experimental prediction is not possible. Discussion papers by BRE (Bourke, K. and Davies, H., 1997), Teplý (Teplý, B., 1999) and others give details of this approach (Aarseth, L. I. And Hovde, P. J., 1999), (Bourke, K. and Davies, H., 1999), (Hed, G., 1998), (Hovde, P. J., 1998), (Leino, T., 1999), (Moser, K., 1999), (RILEM, 1997), (Robertsen, E., 1999), (Strand, S. M. and Hovde, P. J., 1999), (Vesikari, E., 1999).

The Factor Method is included in the ISO (International Organisation for Standardisation) document, ISO 15686 'Service Life Planning' (ISO, 1998). Chapter 7 in this standard indicates the intended level of use for this approach. It states that the Factor Method is a way of bringing together the consideration of each of the variables that is likely to have an effect on service life. Also, it can be used to make a systematic assessment even when exposure data does not fully match the anticipated conditions of use. The idea is that its use can bring together the experience of designers, observations, intentions of the managers, and manufacturers' assurances as well as data from testing institutions.

Aarseth and Hovde (Aarseth, L. I. and Hovde, P. J., 1999) state "The Factor Method allows an estimate of the service life to be made for a particular component or assembly in specific conditions. The method is based on a reference service life and modifying factors that relate to the specific condition of the element." Table 1 describes the factors used in this approach, see also Aarseth and Hovde (Aarseth, L. I. and Hovde, P. J., 1999). The factors have been designed to cover the main aspects affecting the service life. However, this does not mean that in some cases it may prove more suitable to include other relevant modifying factors. The value 1.0 is taken to be neutral, as it will obviously have no effect on the service life calculation. In the ISO 15686 document (ISO, 1998), 0.8 is used for the deteriorating effect, and 1.2 for the favourable conditions.

The relation for the service life prediction may be written as follows:

$$L = L_{ref} \times A \times B \times C \times D_1 \times D_2 \times E \times F$$ **Equation 1**

The reference service life, $L_{ref}$, is the expected service life in the conditions that generally apply to that type of element. Teplý (Teplý, B., 1999) states that it may be determined:

- by means of more accurate methods and approaches,
- based on the producer's data,
- based on the testing laboratory data,
- based on previous experience in similar structures and materials under similar conditions,
- on the basis of agreement in relevant bodies or commissions of the European Union in co-operation with national institutions,
- based on the data in the existing standards and other technical literature.

As discussed earlier, the Factor Method was developed as a tool to support service life prediction in cases where there is a lack of adequate or reliable data. Bourke and Davies (Bourke, K. and Davies, H., 1997) state that it must be used with care and understanding of the method and the project under consideration. However, the following quotation from ISO/CD 15686 (ISO, 1998) indicates the limitations of this method:

"The Factor Method does not provide an assurance of service life. It merely gives an estimate based on what information is available. It is less reliable than a fully developed prediction of service life. The distinction between estimated and a predicted service life should be made when a forecast of service life is given. The information taken into account should also be recorded, so that it is clear whether the estimate is particularly robust or not."

| **Factors** | | **Relevant Conditions (Examples)** | |
|---|---|---|---|
| Agent related to the inherent quality characteristics | A | Quality of components | Manufacture, storage, transport, materials, protective coatings (factory applied) |
| | B | Design Level | Incorporation, sheltering by rest of structure |
| | C | Work execution level | Site management, level of workmanship, climatic conditions during the work execution |
| Environment | $D_1$ | Indoor environment | Aggressiveness of environment, ventilation, condensation |
| | $D_2$ | Outdoor environment | Elevation of the building, micro-environment conditions, traffic emissions, weathering factors |
| | E | In-use conditions | Mechanical impact, category of users, wear and tear |

| | | | |
|---|---|---|---|
| Operating Conditions | F | Maintenance level | Quality and frequency of maintenance, accessibility for maintenance |

**Table 1: Deterioration Factors of Materials and Components (Aarseth, L. I. and Hovde, P. J., 1999).**

Other concerns have been raised regarding this method. These include the possibility of providing misleading output due to the subjective approach used in calculating the factors and the suggestion that this method provides a deterministic value rather than a range or distribution. Also, in relation to the factors used, Teplý (Teplý, B., 1999) states that in certain cases values may be out of proportion. With this in mind, each factor must be carefully addressed to eliminate this ambiguity. Indeed, Aarseth and Hovde (Aarseth, L. I. and Hovde, P. J., 1999) suggest a method to overcome this problem. Each of these criticisms has value, and the method needs to be used with due attention to the changing state of the art of service life prediction.

### 2.1 Developments of the Factor Method Towards a Probabilistic Approach

Since the adoption of the Factor Method several issues have been raised as potential modifications or improvements to the approach. In particular, Hovde (Hovde, P. J., 1998) presents an evaluation of this method and includes suggestions for further studies to reduce uncertainty and improve user-friendliness. In other research, Hed (Hed, G., 1998) presents a project where the Factor Method was assessed. This method has been suggested as an alternate means of estimating service life of components and materials. However, previous use of this method has not been well documented, nor has the development of the various factors used in this method been demonstrated. Only now are these concerns being addressed. As discussed by Aarseth & Hovde (Aarseth, L. I. and Hovde, P. J., 1999), one of the shortcomings in the Factor Method is that the service life is treated as a deterministic value. In reality service life has a large scatter and should be treated as a stochastic quantity (Siemes,T., 1997).

However, more recently, stochastic approaches to the Factor Method have been developed. Moser (Moser, K., 1999) describes this approach for a simple building component. Using probability density functions either from supplier's data, testing or estimates employing variations of the Factor Method, simple examples are provided for this component. However, if this data is not available, a density curve of the estimated service life can be defined by professional estimate or by the recursive Delphi method, see Moser (Moser, K., 1999). This method uses first inputs by experts. After analysis of these results the experts are required to give a modified second input, which is usually more precise. This procedure has given good results in the absence of sufficient data.

Aarseth and Hovde (Aarseth, L. I. and Hovde, P. J., 1999) discuss a similar approach, described as the 'step-by-step' principle. The distribution function is restricted to the Erlang-distribution and based on estimates of most expected value, and the 1% and 99% fractiles for each individual factor. In this way the uncertainty is identified and estimated for these factors. It was suggested that the most uncertain factors should, if possible, be divided into sub-elements and more information gathered so as to reduce this uncertainty. The factors are treated as elements which are summed up to give the estimated service life. Also, these estimates are expressed in years instead of values close to 1. This was done to enable the consequences of each estimate to be seen at each step in the process. However, it proved difficult for experts to assess the estimates. Also, it seems doubtful whether this principle adequately covers the real variation of the individual factors.

Despite the criticisms of the Factor Method, it has not prevented its adoption for use in service life planning and whole life costing. The main reason for this is that no other suitable approach has been proposed. However, additional research is needed for reliable estimation of service life, especially into the reference service lives of structures and their members, but also to recalibrate and refine the modifying factors for different situations.

## 3 ANALYSIS OF DEFECTS IN CONCRETE BRIDGES

The database contains the defect histories of 439 concrete bridges in the UK. The life span of many of these bridges goes back to the early 1930's with the associated inspection records covering most of this period. Indeed, a number of these bridges were constructed as long ago as 1889.

The data collected from the inspection records was organised into three tables. The main categories are given in Table 1 of a paper by Rigden et al (Rigden, S. R., et al, 1993). The defect conditions noted from the inspections are categorised as in Table 2 of the same paper (Rigden, S. R., et al, 1993).

The database package used was Microsoft Access. This has the advantage of being compatible with spreadsheet software so that statistical information can be produced from any analysis undertaken on the database. The tables were viewed to determine which fields could be queried in order to extract useful and meaningful information.

By combining the data fields from each table a further table was produced, see Table 2. A key to the headings used is listed below. This table contains the relevant data used to carry out the analyses.

**Database Query** Code; Year Built; Inspection Year; Type of Structure; Component Type; Exposure Condition; Urgency; Fault/Action Code; Age(years).

| Code | Year built | Inyr | Type | Member | exp | urg | Fault | Age (years) |
|---|---|---|---|---|---|---|---|---|
| TC001 | 1937 | 1966 | 5 | 1 | 2 | 2 | 7 | 29 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| TC001 | 1937 | 1970 | 5 | 1 | 2 | 1 | 11 | 33 |
| TC002 | 1937 | 1970 | 5 | 1 | 3 | 1 | 8 | 33 |
| TC002 | 1937 | 1976 | 5 | 1 | 3 | 1 | 9 | 39 |
| TC002 | 1937 | 1982 | 5 | 1 | 3 | 3 | 9 | 45 |

**Table 2:Typical Layout of Query Table**

The database contains a wealth of information, with some data being more pertinent than others. To determine what data was useful, an extensive analysis of the database was undertaken. To begin, the data was analysed to determine the type of structures and number of members inspected. This gave an overall view of the data stored. The members were then analysed to investigate their degradation over time. The same approach was applied to investigate each individual structure. The relevant results of this analysis were reported by Rigden et al (Rigden, S. R., et al, 1993), (Rigden, S. R. et al, 1996) and Mc Parland et al (Mc Parland, C. B. et al, 2001).

Table 3 and Table 4 display the age at which the structure/member fails (as defined by the need for major repair) under the given conditions. They are a summary of the information obtained through preliminary analysis of the database. A number of significant trends became apparent through analysis of the database. The first step was to obtain a general view of the rate of degradation of all members that had been inspected over the last century. It was found that the majority of members had reached the end of their service life after just over 40 years (Service Life is reached when either failure occurs or major repair is necessary to keep the structure in service).

| | **Exposure 1 (Years)** | **Exposure 2 (Years)** |
|---|---|---|
| **Flexural** | 42.5 | 34.7 |
| **Compression** | 45.1 | 36.3 |

**Table 3: Service Failure of Members under (1) Mild and (2) Moderate Exposure.**

| | **Exposure Ignored (Years)** | **Exposure 1 (Years)** | **Exposure 2 (Years)** | **Flexural (Years)** | **Compression (Years)** |
|---|---|---|---|---|---|
| **Footbridge** | 40.1 | 43.1 | 37.0 | 37.9 | 43.5 |
| **Overbridge** | 27.8 | 26.4 | 26.5 | 33.1 | 20.4 |
| **Underbridge** | 45.4 | 53.5 | 39.2 | 45.2 | 51.1 |

**Table 4: Service Failure of Structures**

Various structures were analysed individually in order to determine the rate of degradation and make comparisons. Exposure conditions had obvious detrimental effects on the overall structure. Perhaps one of the most interesting findings of the analysis is the fact that under-bridges are most likely to perform longer than any other bridge structure. The reasons for this may lie in the construction type and the form of transport using these bridges; an under-bridge is a structure in which vehicles pass under.

The main cause of deterioration is most likely to be due to corrosion caused by chloride attack or carbonation. Much of the most severe concentrations of chloride levels from de-icing salt occur where bridge deck joints have leaked, allowing chlorides in water to seep through to the substructure. Rigden et al (Rigden, S. R. et al, 1996) reported that the main cause of defects in concrete bridges was due to leaking deck joints. Consequently, abutments and piers become contaminated and severe corrosion result. Spray from passing traffic can cause chloride contamination of roadside piers and abutments, but the concentrations tend not to be as high as that from defective deck joints. Members were analysed according to the structure type they were part of, flexural and compression members followed similar trends of degradation with the former deteriorating faster (Mc Parland, C. B. et al, 2001).

From the results obtained by the interrogation of the database it became apparent that there was a definite pattern in the way the information changed under certain conditions (Mc Parland, C. B., et al, 2001). It confirmed the belief that, for a given structure or member, the age and subsequent exposure conditions were key factors in determining the deterioration rate of the element. Using these factors it may then be possible to develop a model that could readily predict the deterioration rate of a given concrete structure.

## 4 DEVELOPMENT OF FACTOR METHOD USING A SPREADSHEET

The Factor Method has been discussed in previous sections. The processes involved in determining the estimated service life, L, of a structure are detailed. In earlier models factors were calculated based on deterministic rules, empirical methods or by subjective judgement. Later models attempted to use probabilistic approaches to determine the relevant factors. However, no ‘actual’ data, collated over the past decades, were ever used. The development of the Factor Method using a spreadsheet is based on this approach. The relevant factors used in the adapted Factor Method are calculated by simple probabilistic methods using data from the database.

In the spreadsheet program, there are two forms. One is used to calculate the factors whenever the overall structure is being considered. The second form is used whenever individual members of a structure are analysed. Details of their development are given in the following sections.

### 4.1 Deterioration Rate of Overall Structure

Deterioration rate of Overall Structure is evaluated using Form 1. Table 1 of Form 1 displays the age at which a structure is most likely to deteriorate to a specific stage in its life cycle. These stages can either be defect free (1 – 3), cracking (4 – 6), spalling (7 – 9) or service failure (10 – 12). The values listed in this table are calculated from all the available data contained in the database. These will be used as the reference service life, $L_{ref}$, as mentioned in the Factor Method. To determine the expected service life, L, of a given structure two factors are first calculated. These are factors $F_1$ and $F_2$. Factor $F_1$ is based on the structure in question, as each structural type deteriorates at a different rate. This factor is calculated as follows:

- First, the database is queried so that only data related to the specific structure is used in the calculation.
- Next, the selected data is filtered into four categories depending on which stage of deterioration they relate to.
- The average and standard deviation of each category is calculated.
- The averages are then divided by the relevant values of $L_{ref}$, (Appendix A: Form1, Table 1).
- The resulting factors are averaged to provide an overall factor, $F_1$.

Factor $F_2$ is calculated in a similar way. The only difference is that the data from the inspection records is filtered depending on both structural type and exposure condition.

The final stage in the process is calculating the expected service life, L. This is done by multiplying each average in Table 1 of Form 1 by both factors $F_1$ and $F_2$. The resulting values are displayed in Table 2 of this form.

The overall process involved in determining the expected service life of individual members is similar to that previously mentioned (See Form 2). The major difference is that there are now three factors to be calculated. Factor $F_1$ is based on the structural type, $F_2$ is based on the member type and $F_3$ is based on the exposure condition. As before, the reference service life for each phase is multiplied by the appropriate values of $F_1 – F_3$ to provide the expected service life.

## 5 DISCUSSION OF THE RESULTS

The adaptation of the Factor Method using a spreadsheet offers a number of benefits to the user. The advantages of using a spreadsheet to develop this model can be listed as follows:

- It is user-friendly. Someone with limited knowledge in this field could competently use this approach. Other models of the Factor Method have proved to be quite difficult to use because they are often based on complex mathematical equations and distributions.
- Once the appropriate data has been entered the relevant information is displayed immediately. This avoids the need to search the database.

Other advantages in using this approach are apparent:

- There is a substantial amount of data available to provide reasonably accurate results.
- As more data is made available, the likelihood of obtaining a more accurate and consistent set of results increases. Also, with the availability of more data, more factors, such as those used to describe operating conditions, may be calculated. This would increase the overall effectiveness of the model.

With the development of a model, there will always be a number of shortcomings reducing its overall effectiveness. These include:

- Although there is a large amount of data that can be extracted from the inspection records, in some cases the amount of information available to be used is limited for certain types of structures.
- The data needs to be analysed further to remove records where unusual events occurred.
- In some cases, a data set containing relatively few values was used to determine a factor. Such factor would be sensitive to ambiguous data.
- For each stage in the deterioration process a factor was calculated. These four values were then averaged to provide an overall value for the factor. This process dilutes the effectiveness of the factor. This was due to insufficient data being available to calculate individual factors for each stage of the life cycle. The above

method provides an overall value to be used in this case. Whenever more data is made available, then this approach will be reviewed.

## 6 CONCLUSIONS

This paper highlights the need to model the deterioration process of concrete structures. The age and exposure conditions of concrete structures are important as part of the modelling process. They can be used to provide a classification of the deterioration of a structural element along with the associated urgency of maintenance required.

Following the development of the Factor Method, many researchers have considered different approaches to incorporate it into design and maintenance programmes. Previous models that were developed proved to be complex; it is believed that this problem has been overcome to an extent by using a spreadsheet.

On the negative side, the classification is only as accurate and suitable as the data provided by the previous records. Although it may not be the most accurate model currently available, it does provide realistic results. As more records are made available then the accuracy and relevance of the information for the structure and / or component will become more prevalent.

Overall, the model developed provides a useful tool to the engineer when an inspection is to be carried out. The information shows what to expect before the inspection begins. It will also help the engineer in determining the most appropriate maintenance and repair techniques to use.

# Form 1: Deterioration Rate of Overall Structure

*From analysis of the database, the age at which a structure is most likely to deteriorate to a specific phase is given below.*

| Phase | Age (Years) | s.d. |
|---|---|---|
| Defect Free | *25.5* | *18.6* |
| Cracking | *31.0* | *17.5* |
| Spalling | *37.5* | *18.0* |
| Failure | *39.9* | *21.2* |

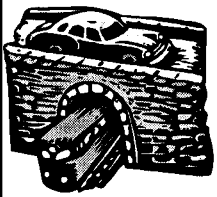
**Table 1** Values of $L_{ref}$ for each Phase

**Note: The age will be used as the Reference Service Life, $L_{ref}$**

*Please Complete the Following Sections*

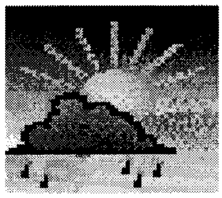
**Section 1: Structure Type**

| | Structure |
|---|---|
| 1: | *Footbridge* |
| 2: | *Overbridge* |
| 3: | *Underbridge* |
| 4: | *Others* |



**Section 2: Exposure Condition**

| | Exposure |
|---|---|
| 1: | *Mild* |
| 2: | *Moderate* |
| 3: | *Severe* |



1

**Enter Corresponding Number in Box Provided.**

3

*Output*

**Footbridge subject to severe exposure.**

**The following output is likely:**

**Calculated Factors**

| Factor | Value |
|---|---|
| $F_1$ | *1.01* |
| $F_2$ | *1.32* |

**Note: Expected Service Life, L, is calculated as follows:**

$$L = L_{ref} \times F_1 \times F_2$$

**Expected Service Life, L**

| Phase | Age (Years) |
|---|---|
| Defect Free | *34.0* |
| Cracking | *41.3* |
| Spalling | *50.0* |
| Service Failure | *53.2* |

**Table 2** Expected Time to each Phase.

## Form 2: Deterioration Rate of Individual Members

*From analysis of the database, the age at which a member is most likely to deteriorate to a specific phase is given below.*

| Phase | Age (Years) | s.d. |
|---|---|---|
| Defect Free | *25.5* | *18.6* |
| Cracking | *31.0* | *17.5* |
| Spalling | *37.5* | *18.0* |
| Failure | *39.9* | *21.2* |

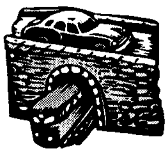
**Table 1** Values of Lref for each Phase

**Note: The age will be used as the Reference Service Life, $L_{ref}$**

### *Please Complete the Following Sections*

**Section 1: Structure Type**

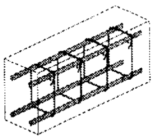
| | Structure |
|---|---|
| 1: | *Footbridge* |
| 2: | *Overbridge* |
| 3: | *Underbridge* |
| 4: | *Others* |



2

**Section 2: Member Type**

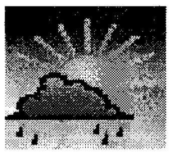
| | Member |
|---|---|
| 1: | *Flexural* |
| 2: | *Compression* |
| 3: | *Others* |



2

**Section 2: Exposure Condition**

| | Exposure |
|---|---|
| 1: | *Mild* |
| 2: | *Moderate* |
| 3: | *Severe* |



3

**Enter Corresponding Number in Box Provided.**

### *Output*

**Compression Members from an Overbridge subject to Severe Exposure.**

**The following output is likely:**

**Calculated Factors**

| Factor | Value |
|---|---|
| $F_1$ | *0.87* |
| $F_2$ | *0.80* |
| $F_3$ | *No Data* |

**Note: Expected Service Life, L, is calculated as follows:**

$$L = L_{ref} \times F_1 \times F_2$$

**Expected Service Life, L**

| Phase | Age (Years) |
|---|---|
| Defect Free | *17.7* |
| Cracking | *21.6* |
| Spalling | *26.1* |
| Service Failure | *27.8* |

**Table 2** Expected Time to each Phase

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# Lcaid™ Software: Measuring Environmental Performance Of Buildings

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**Summary**: **With a move towards achieving sustainablility in the construction industry, there is an increasing need to measure the environmental performance of buildings. At present many buildings are claiming to be "green" but the basis of these claims are often unclear, subjective or narrow in focus. What is needed is an objective and quantitative measure of performance across a range of environmental indicators. One possibility is Life Cycle Assessment (LCA), an internationally accepted environmental accounting system of all the inputs and outputs of a product, material or building. This paper looks at LCAid™, a life cycle assessment software developed by the NSW Department of Public Works and Services (DPWS) that provides quick, comprehensive, and scientifically based environmental assessment of buildings. LCAid™ evaluates environmental performance, identifies impacts against eleven eco-indicators and separates impacts into four stages of the building's life cycle. It is aimed at the building designer as a user friendly decision-making tool for evaluating the environmental performance and impacts of designs and options over the life cycle of a building, product or system. The software has a number of unique capabilities: it can import material quantities from CAD drawings, can import LCI data directly from Boustead and SimaPro and can compare up to 5 design options at once. A Life Cycle Costing module has been completed, which adds an economic dimension to the comparison, as a monetary value. LCAid™ is an easy-to-use tool that aims to help move the Australian building and construction industry towards more sustainable practices.**

**Keywords. Sustainable Buildings, Life cycle assessment, LCAid™**

## 1 INTRODUCTION

Buildings are major consumers of resources both in their infrastructure and in their operation. This is a major concern in achieving sustainability, "according to Worldwatch Institute building construction consumes 40 percent of the raw stone, gravel and sand used globally each year and 25 percent of virgin wood. Buildings also account for 40 percent of the energy and 16 percent of the water used annually worldwide" (Lippiat 1998). As noted by the US National Pollution Prevention Centre (1999) "the construction and operation of buildings consume the majority of the world's natural resources and energy, and contribute the bulk of landfill waste "and this is supported by CIRIA (2001), "construction activities generate waste flows on a large and unsustainable scale."

The built environment contributes significantly to overall environmental degradation. By improving design and construction of buildings we can reduce this impact (DPWS 2001). Addressing the environmental performance of building materials through their life cycle is a crucial factor in achieving sustainable buildings. This is supported by Lippiat and Norris (1996) who state that "selecting environmentally preferable building materials is one way to improve a buildings environmental performance." The built environment plays a vital role in both in human health and well-being as well as in achieving sustainable development.

The issue of sustainable buildings has been achieving recognition as an important sustainable development issue and is clearly a matter of worldwide concern. (Remkes 2000). This is supported by Zachmann (2000), "sustainability is also for construction and our built environment, one of the biggest challenges for the 21$^{st}$ century". Parts of the construction industry have also embraced the challenges and opportunities offered by sustainability believing that "anticipating change and responding to the wider sustainability agenda is critical to long term competitiveness" (CIRIA 2001).

The way buildings are measured for performance has been changing. Over the past few decades, environmental factors have played an increasing role in how a building's performance is measured. With the realisation that many practices within the building and construction industry are unsustainable and a heightened awareness of the environmental impacts of materials, products and buildings there has been a move to address the economic, social and environmental performance of buildings.

Many new projects claim to have improved environmental performance but the basis of these claims is often unclear or narrow in focus. To substantiate these claims a measurement methodology and tools are required that are objective and cover a wide range of environmental impacts. Life cycle assessment (LCA), an internationally accepted environmental assessment methodology is a possibility. LCA encapsulates both the operational resource consumption of a building and the material resource consumption from resource extraction through to final disposal.

## 2 LIFE CYCLE ASSESSMENT (LCA)

Life cycle assessment is defined as the "compilation and evaluation of the inputs, outputs and the potential environmental impacts of a product system through its life cycle" (Australian/New Zealand Standard (no 14040) 1998). LCA is endorsed by the international standards association and criteria for LCA are contained in the ISO 14040 series. LCA is defined in ISO14040 as "a technique for assessing environmental aspects and potential impacts associated with a product by:

- compiling an inventory of relevant inputs and outputs of a product system;
- evaluating the potential environmental impacts associated with those inputs and outputs;
- interpreting the results of the inventory analysis and impact assessment phases in relation to the objectives of the study."

LCA evaluates resource use and emissions of a product in a cradle-to-grave approach studying potential environmental impacts from raw material extraction, through manufacturing, construction, operation, demolition and disposal.

LCA has been used to improve process and product design and provide data for benchmarking. In designing the built environment LCA can provide both designers and specifiers with quantitative options for integrating environmental improvements, seeking environmentally friendly alternative materials and substantiating environmental performance claims. In this way, LCA can be used as a tool to promote and achieve sustainable development, particularly in the design and construction industries by providing quantifiable comparisons between various design and construction options, as well as between different buildings by way of benchmarking.

Environment Australia commissioned a project consortium led by the Centre for Design at RMIT to "assess the status of life cycle assessment (LCA) tools in the building and construction (B&C) industry and to develop strategies to improve the uptake and use of these tools". The aim of the project was to "improve the environmental performance of the B&C industry, by promoting LCA as a tool to assess environmental impacts of building materials and building systems in Australia." (Centre for Design RMIT, 2001). This study highlighted the absence of a suitable commercially available LCA tool in Australia for building simulations, similar to tools available elsewhere in the world.

DPWS has undertaken extensive LCA research in the past decade. This work can be divided into 2 distinct periods. Prior to 1999 work focused on LCA research and development of DPWS Life Cycle Inventory (LCI) database using the Boustead Model. In the period 1995 to 1999 DPWS had a group of specialists consisting of environmental engineers and chemists who carried out extensive research into the life cycle performance of materials and buildings. The database created of materials consisting of over 100 LCIs and over 200 composite building materials are considered one of the most extensive in Australia. After 1999 work moved from research to application, focussing on the development and use of LCAid™ software. There was recognition of the need for a quick, easy and effective LCA tool for designers and architects – a tool that easily fits into the design process. LCAid™ was conceived as a designer's LCA tool.

## 3 LCAID™

LCAid™ is a computer software developed by NSW Department of Public Works and Services (DPWS) with Dr. Andrew Marsh (formerly of University of Western Australia and now with Cardiff University, Wales). The softwares aim is to make LCA more accessible to design and building practitioners for environmental assessments and design improvements. As noted by Hall and Peshos (2001) "LCAid™ arose from the need to provide a fast, comprehensive and scientifically based environmental assessment of buildings. It is aimed at the building designer, and is a user friendly decision making tool using LCA methodology to evaluate the environmental performance of design options and to identify the largest impacts over the entire life cycle of a building".

LCAid™ uses quantities of building components combined with operational energy and water consumption to calculate environmental outputs. These are calculated over eleven eco-indicators covering atmospherics, resources and pollutants. It also separates the environmental impacts within each eco-indicator into four stages; construction, operation, maintenance and demolition. The following diagram illustrates the environmental issues and scope considered by LCAid™.

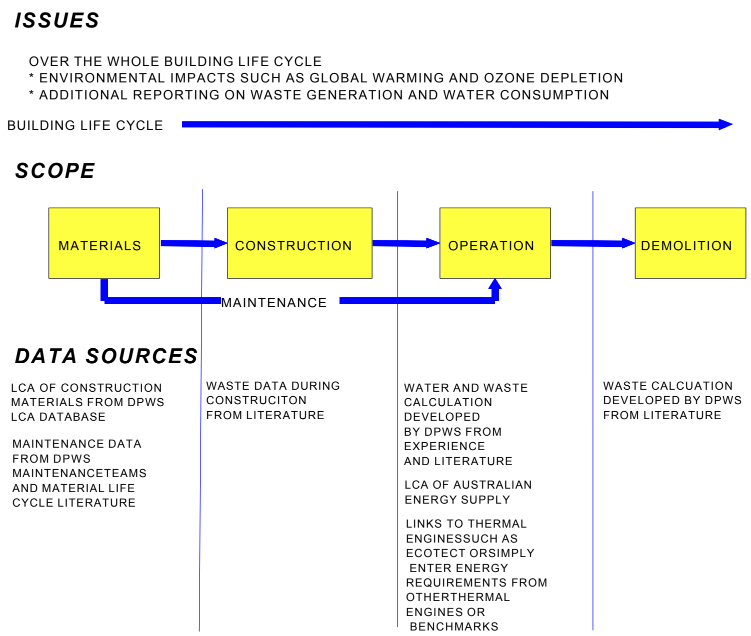


**Figure 1. LCAid™ Scope and Issues**

## 3.1 Inputs

### 3.1.1 Project and Operational Data

Project and operational input is required covering the areas of project type, climate zone, operational energy, waste management, water management and water use. This data is used to perform specific calculations for issues such as waste generation and water consumption. Figure 2 demonstrates one of the user input screens.

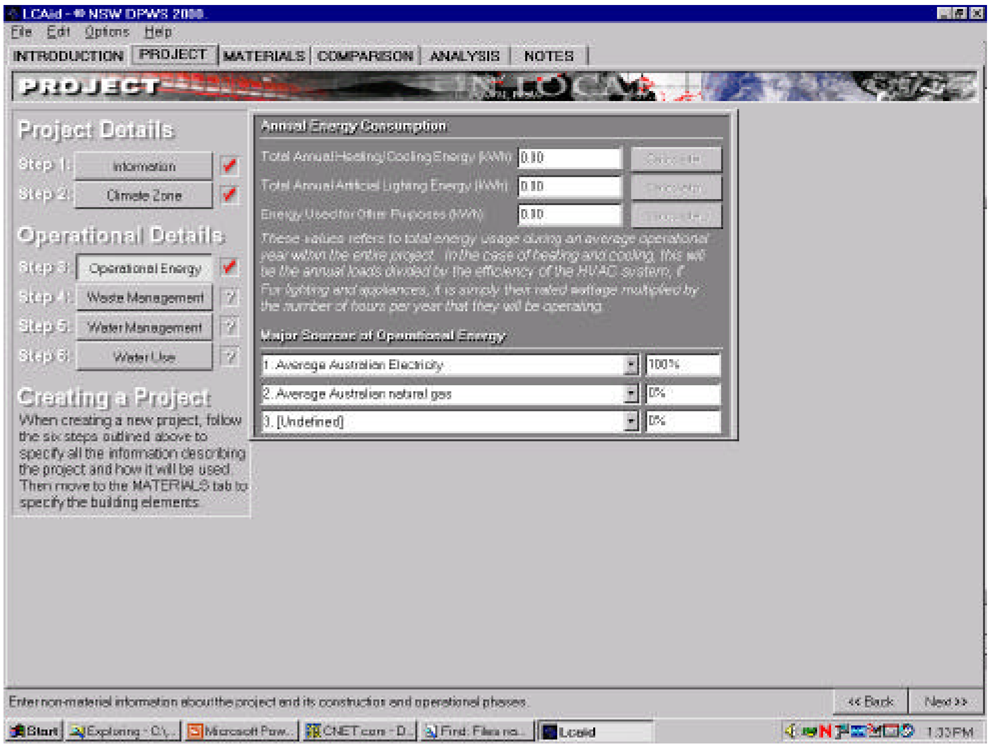


**Figure 2. Project and Operational Information user input screen**

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### 3.1.2 Building Materials

Building material quantities and types can be entered in a number of ways. A unique feature of LCAid™ is that building material quantities can be imported from 3-D architectural models through CAD drawings (dxf) and Ecotect (.eco or .zon) models. As noted by Hall & Peshos (2001) "this link to CAD packages provides the basis through which LCAid™ can be integrated into the design process" and the software's "power stems from its seamless integration with other environmental software - it can work on a 3D model created in Ecotect" (Hall & Peshos 2001).

Building materials quantities extracted from the 3-D models or manually entered by the user are then assigned material types by selecting from the LCI library as shown in Figure 3 below. The LCAid™ library draws upon the DPWS LCA database consisting of over 200 composite common Australian building materials. LCAid™ can import LCI data directly from other LCI databases including Boustead Model and Sima-Pro.

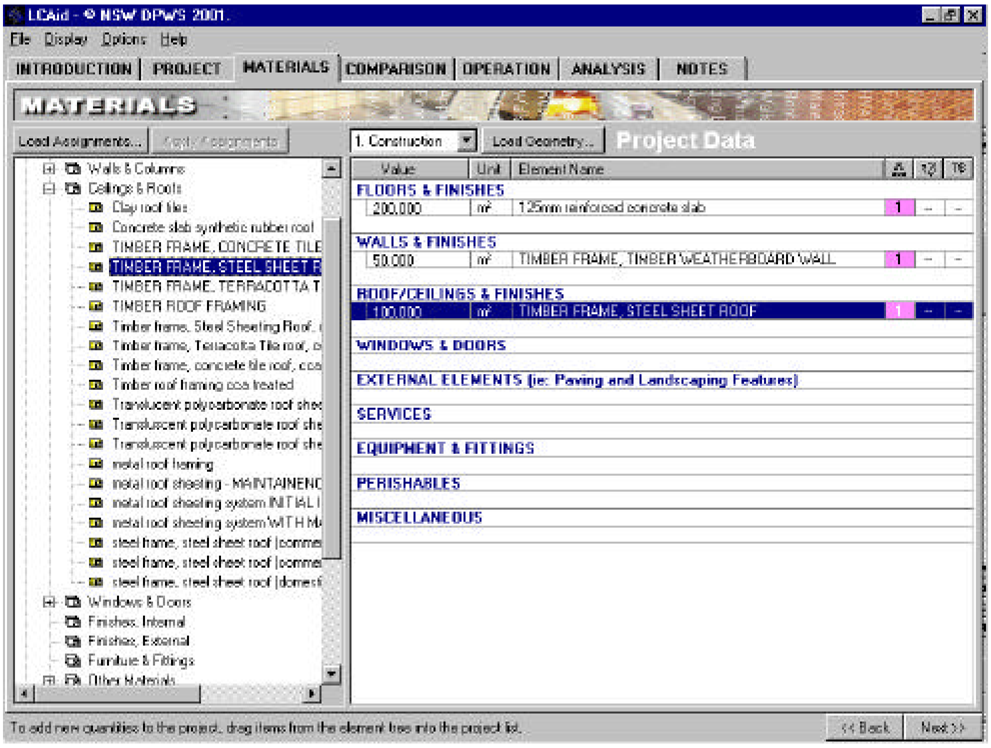


**Figure 3. Materials Input Screen**

## 3.2 Outputs

LCAid™ currently has two main outputs, reporting the environmental impacts of a building in two different ways. Firstly the environmental performance is determined by comparing a design to a benchmarked building and the tool also identifies the environmental impacts for a design at each stage of the buildings life cycle (Hall & Peshos 2001).

### 3.2.1 Environmental Comparison

For each design the environmental outputs are graphed with quantities shown for each Eco-indicator, as shown in Figure 4. The data is calculated using the impact categories of Eco-indicator 95 with the additional reporting of water and solid waste. There is the potential for the impact categories to be expanded to include eco-indicator 99 and other environmental impacts such as biodiversity and indoor air quality.

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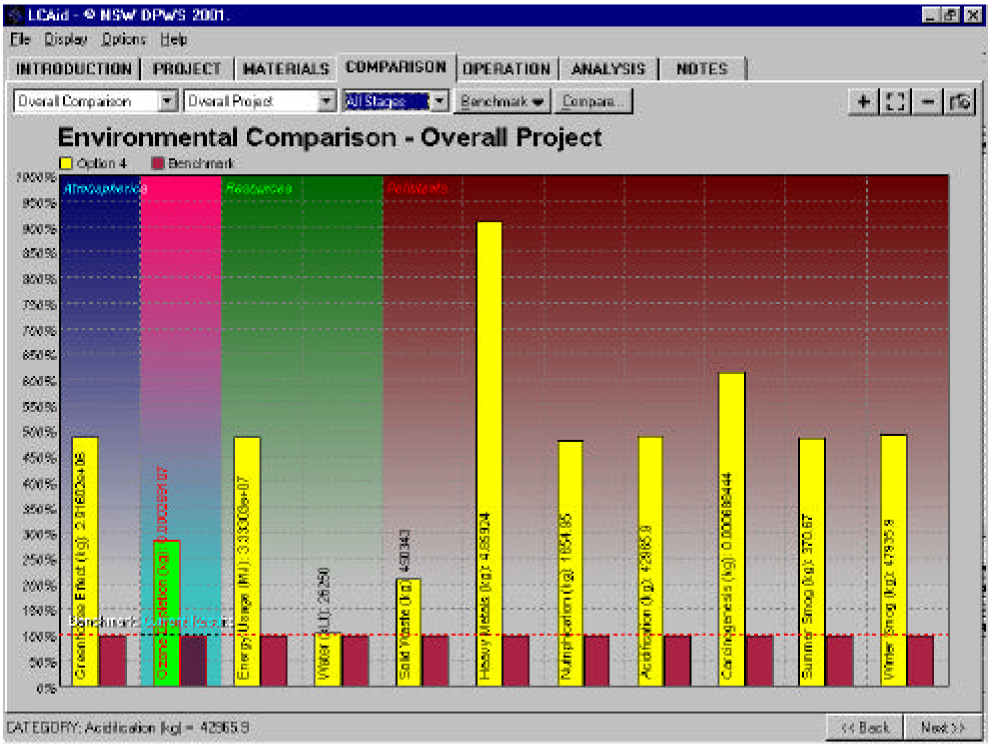


**Figure 4. Environmental Impacts**

As a comparative tool, a design can be benchmarked and other design options can be compared against it. This allows the designer to assess the performance of an option relative to a determined benchmark and provides data for informed decisions to be made. LCAid™ can compare up to 5 options at once as shown in Figure 5.

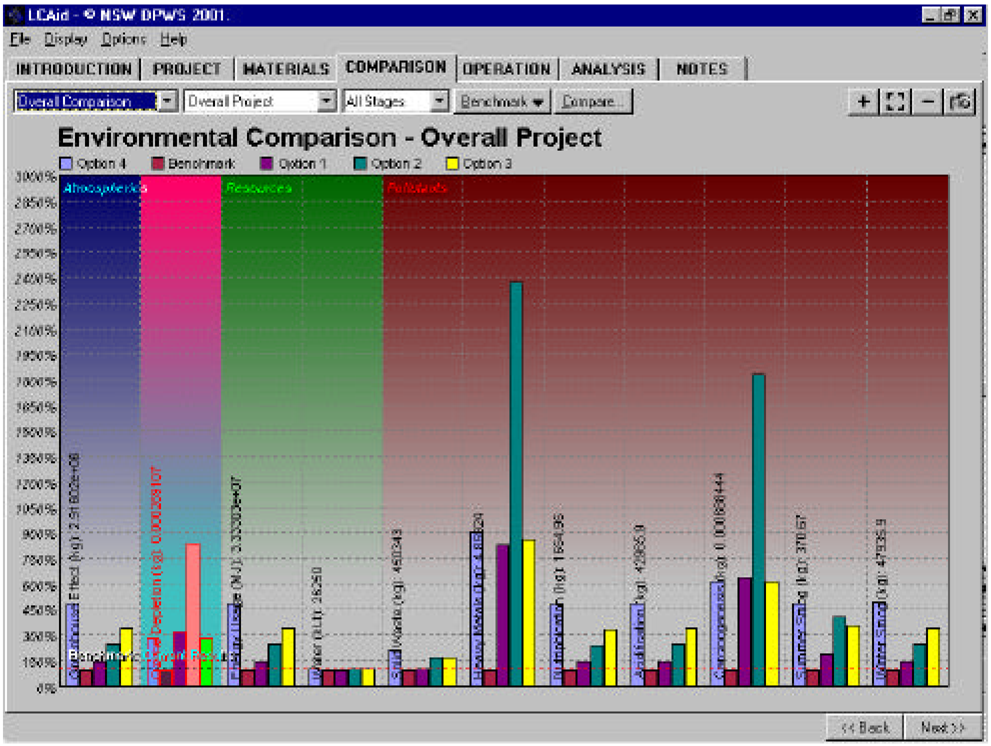


**Figure 5. Comparison of Options**

### 3.2.2 Environmental Impacts

For a design option the contribution of each stage of a building's life is shown for each eco-indicator. The impacts from construction, operation, maintenance and demolition are indicated and for each eco-indicator the materials contributing are ranked. Their contribution of the total is identified as shown in Figure 6 below. This allows the user to easily identify materials or combinations of, which are of greatest potential impact, allowing the user to easily identify areas for improvement.

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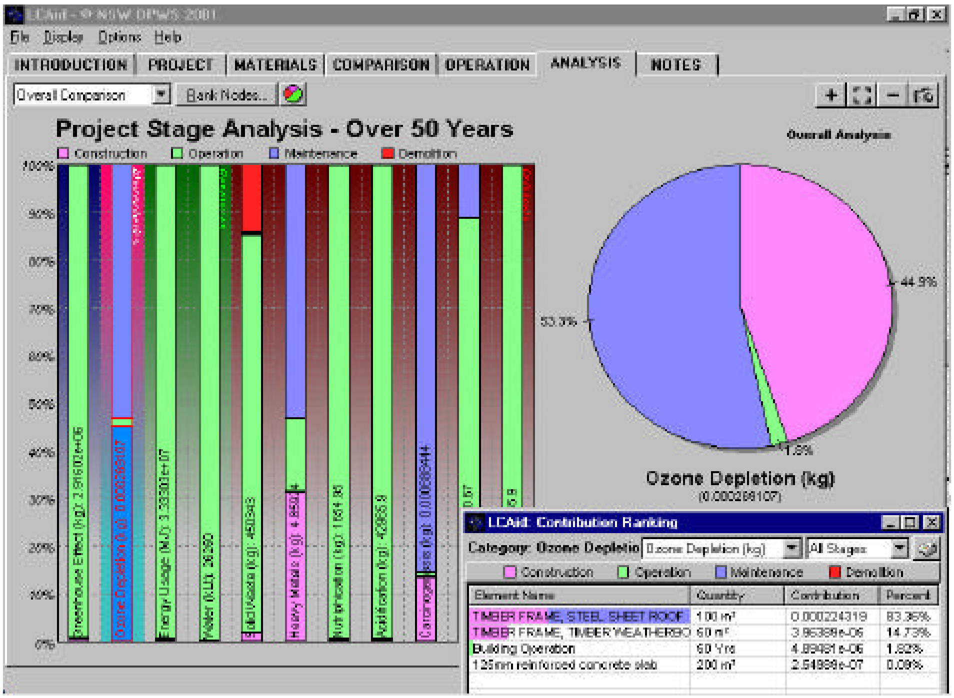


**Figure 6. Project Stage Analysis and Materials Contribution Ranking**

### 3.3 Life Cycle Costing Module

A life cycle costing (LCC) module has been completed, which adds an economic dimension to the comparison, as a monetary value. Life cycle costing is a mathematical method of measuring the life costs of a building. It takes into account ongoing maintenance and replacement costs over the full life of the project, not just the initial capital outlay. This allows the users to fully assess the initial costs of a venture as well as the ongoing maintenance and operation against the environmental performance, although it is not possible to put a monetary cost on all environmental impacts. As noted by Lippiat and Norris (1996) "Even the most environmentally conscious building designer or building materials manufacturer will ultimately want to weigh environmental benefit against economic costs". The data for the costing module has been provided by the DPWS Quantity Surveying section, who have collected data on building and material costs and maintenance regimes. The LCC methodologies used are a hybrid of *Nett Present Value* and *Internal Rate of Return* methods.

LCAid™ uses the materials and operational data provided to graph the life cycle cost of building options. Figure 7 following shows the life cycle costs display.

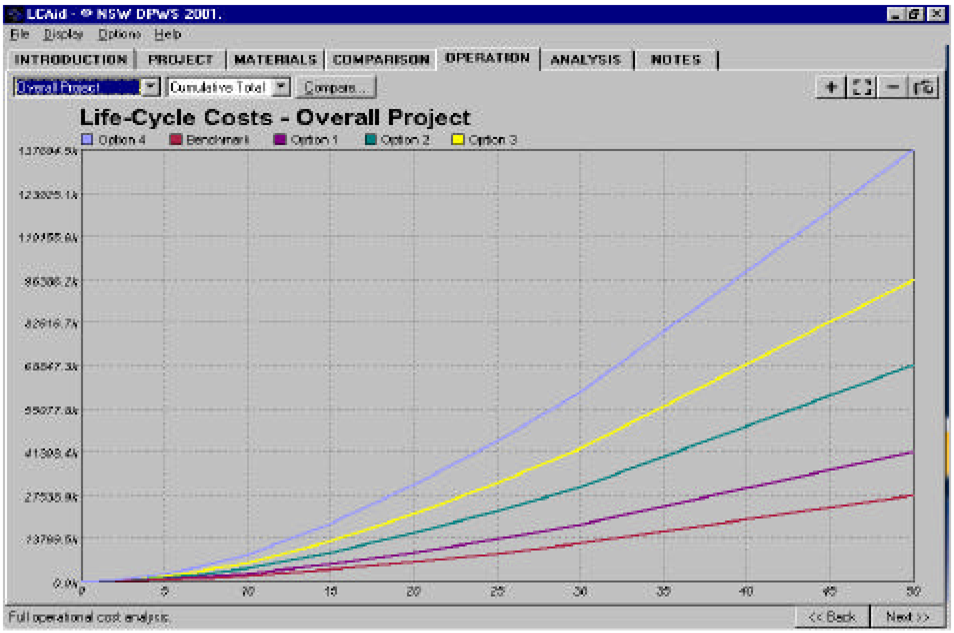

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**Figure 7. Life Cycle Costing Display**

## 4 ROLE AND BENEFITS OF LCAID™

As a tool to measure environmental performance, particularly at the design stage, the strength of LCAid™ is that it allows the user to gauge where design improvements can be made and how an option performs against a benchmark. LCAid™ is an easy to use tool, providing information on the relative impact of options and allows the comparison of single materials, complex composites and whole buildings. As stated in the background report for the *Greening the Building Life Cycle* project undertaken by RMIT for Environment Australia this idea is supported: "LCA tools are most useful in the design stage of the life cycle" (Centre for Design at RMIT 2001).

In addition to this, LCAid™ can measure both built and proposed buildings, allowing benchmarking between buildings, an important factor for building owners and potential leasees, especially with the inception of performance standards for the awarding of government leasing contracts. LCAid™'s strength is as a design tool where the informed environmental decisions can be made allowing the overall environmental impact to be reduced.

## 5 CONCLUSION

The construction industry's performance is increasingly being measured in environmental terms and as environmental systems become more sophisticated, this performance is increasingly measured over the total life cycle of the product. Building materials and systems can and do have significant impacts on the environment throughout their life cycle and there is a need to measure and quantify these environmental impacts, particularly in the design phase of a building. Life cycle assessment is an environmental methodology for assessing the potential impacts of a material, product or building and can provide the building and construction industry and in particular designers, tools to move towards more sustainable practices. LCAid™ a LCA software aimed at the building designer is an easy to use tool that provides quantifiable information on potential environmental impacts, allows for design options to be compared against a benchmark and provides information to allow for informed environmental decisions to be made.

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# Whole Building LCA With WLC: A New Commercial Software Development For Product Specification In The UK

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**Summary**: **In 2000, BRE launched a software tool called Envest. This tool allowed designers to gauge the environmental impacts of a whole building design at the initial stage of the design process, comparing the embodied impact of the building fabric to the impact from operation. The simplicity of the tool hinges upon the use of a single score approach to measuring environmental impacts. The measure is known as Ecopoints. In response to pressure from the market, BRE has developed Envest further to include a Whole Life Costing function that enables the designer to consider the costs of the building delivery and its maintenance scheme together with the environmental impact. Acknowledging that there is a role for generic and specific costs, the new tool will enable users to enter their own costs or use default data. In early 2002, Envest II will go on line. Web based, the tool will be inherently more flexible than its CD ROM predecessor and, for the first time, creates the possibility of adding new products as data becomes available from manufacturers. This function has been developed in parallel with the BRE Certification Scheme for Environmental profiles of Construction Products which allows manufacturers to make independently verified claims about their product. Envest scores can already be used to obtain "material" credits in BREEAM, the whole building assessment scheme and the new version will continue to be used in this way.**

**Keywords. Construction products, Embodied impacts, Software tool, LCA, WLC, Ecopoints.**

## 1 INTRODUCTION

The integration of Whole Life Costing and Life Cycle Assessment presents a powerful route to improving the sustainability of the construction industry. Combining economic and environmental assessment tools to obtain "best value" solutions in both financial and environmental terms has the potential to make a significant contribution to achieving sustainable building design.

In 2000, BRE launched a software tool called Envest. This tool allows designers to gauge the environmental impacts of a whole building design at the initial stage of the design process, comparing the embodied impact of the building fabric to the impact from operation. The simplicity of the tool hinges upon the use of a single score approach to measuring environmental impacts. The measure is known as Ecopoints. In response to pressure from the market, BRE has developed Envest further to include a Whole Life Costing function that enables the designer to consider the costs of the building delivery and its maintenance scheme together with the environmental impact. Acknowledging that there is a place for generic and specific costs, the new tool will also enable users to enter their own costs data. As a web based software, the tool will be inherently more flexible than its predecessor and opens up the possibility of adding new products as data becomes available from manufacturers.

This paper begins with definitions of Whole Life Costing and Life Cycle Assessment as they have been used in the development of the software. It goes on to outline the concept of Ecopoints. This is followed by the rationale behind the Envest software and a brief overview of the activities required to integrate WLC into the tool.

## 2 DEFINING THE TOOLS

'Whole life costing', 'life cycle costing' and 'life cycle assessment' are terms often used interchangeably. This creates a great deal of confusion. For clarity, user friendly definitions are provided in Box 1. These definitions are those used by BRE and which are commonly used within the industry.

WLC and LCA in the construction industry have developed separately in response to economic and environmental issues but the two tools have much in common, as shown in figure 1.

The key similarity is that both utilise data on:

- quantities of materials used,
- the service life the materials could or will be used for,
- the maintenance and operational implications of using the products,
- end of life proportions to recycling (and sale value) and disposal.

The key differences are:

- conventional whole life costs methods do *not* consider the process of making a product, they are concerned with the market cost. Life cycle assessment considers production.
- WLC is usually discounted to present value over time, environmental impacts are not.

## 2.1 Progress to date

A BRE study of Whole Life Costing conducted for DETR (Clift and Bourke 1999) found that despite substantial amounts of research into the development of database structures to take account of performance and whole life costing, there remains a significant absence of standardisation across the construction industry in terms of scope and data available. The Centre for Whole Life Performance at BRE is working to improve this situation by working with the industry and continuing to develop its own independent whole life cost and performance data.

**Box 1: Common definitions**

**WHOLE LIFE COST (WLC)**

*The definition from the developing ISO Standard 15686 on service life planning is* "a tool to assist in assessing the cost performance of construction work, aimed at facilitating choices where there are alternative means of achieving the client's objectives and where those alternatives differ, not only in their initial costs but also in their subsequent operational costs."

- WLC includes the systematic consideration of all relevant costs and revenues associated with the acquisition, use and maintenance and disposal of an asset.
  - *Procurement costs can include*: initial construction, purchase/lease, interest, fees
- *Recurring costs can include:* rent, rates, cleaning, maintenance, repair, replacement/renewal, energy and utilities, dismantling or disposal, security and management.
  - *Revenues can include:* Sales of recycled materials, interest in asset and rental income.

Note: **Life Cycle Cost** (LCC) and **Through Life Cost** (TLC) are also terms used to describe the same process as WLC. The term is now less commonly applied and therefore WLC is used throughout this document.

**LIFE CYCLE ASSESSMENT (LCA)**

- A method to measure and evaluate the environmental burdens associated with a product system or activity, by describing and assessing the energy and materials used and released to the environment over the life cycle. The term **life cycle analysis** is also sometimes used to describe the same process.

**ENVIRONMENTAL IMPACT ASSESSMENT**

- The process of interpreting the effect that removals or releases to the environment will have on particular environmental systems. There is no absolute "end point" at which an impact has to be measured-e.g. burning coal will cause "fossil fuel depletion", "global warming potential" and even "risk of drought"-but common endpoints and measurement techniques may be agreed which allow the impacts of different activities to be compared.

**ENVIRONMENTAL PROFILE**

- A presentation format for the results of an LCA study. BRE has produced an industry-agreed method for collecting, interpreting and presenting the data according to a standard Environmental Profile format.

**SERVICE LIFE**

- Life of a product or building element or whole building. May be

Technical (based on physical durability and reliability properties),

Economic (based on value and depreciation to owner) or

Obsolescence (based on factors other than time or use patterns e.g. fashion).

Note: In practice, **Replacement Interval/Life** is also used interchangeably with service life and the same distinctions apply.

LCA

**2.2 Stages in the Building Life Cycle**

1. Extraction of raw materials
2. Production of building components
3. Construction
4. Use:
   Repair
   Maintenance
   Replacement
5. Demolition
6. Recycling

WLC

**Figure 1. Commonality within the Building Cycle of issues relating to WLC and LCA**

It is recognised that whole life cost data may not always be readily available and comparisons can be difficult. The concept of option investment appraisal using cost data is however, relatively straight forward. Comparisons on the basis of environmental information have been less easy to make because of the wide range of data available. Simplification of the diverse issues considered has been required - and this has been delivered through the development of a single score for environmental impacts, known as Ecopoints.

Life cycle assessment (LCA) analyses all the environmental impacts from a product cradle to grave. It is now an internationally established analysis technique applied in many industries, including construction. To obtain a single score, such as an Ecopoint, it is essential to start with a sound approach to environmental assessment.

BRE's 'Methodology for Environmental Profiles of construction materials, components and buildings' (Howard, Edwards and Anderson, 1999) provides a standardised way of carrying out LCA on UK construction products. The BRE methodology covers the extraction, processing, manufacture, transport, use and disposal stages of the product's life cycle. It summarises the environmental impacts arising from these stages into thirteen impact categories including climate change, atmospheric and water pollution, and raw materials consumption.

## 3 ECOPOINTS

Comparisons between Environmental Profiles are always informative, but do not necessarily allow the reader to reach a precise conclusion. For example, which gives less overall environmental impact: a product with high global warming impact but low water pollution impact or a product with low global warming impact but causing significant water pollution?

BRE undertook a consensus based research programme to weight the issues covered by LCA from the perspective of seven UK construction interest groups, including the public sector, construction materials producers and manufacturers, property professionals, environmentalists and academics.

The results showed a surprising degree of consensus about the relative importance of different environmental issues across a broad range of interest groups. This consensus has been used to produce a set of weights to convert Environmental Profiles data into a single score reflecting environmental impact in the UK. The data in the twelve impact categories are multiplied by the agreed weight for each category and combined to produce an Ecopoint score.

**Box 2. Environmental impact categories**

| | |
|---|---|
| **Climate Change** | **Pollution to Water: Ecotoxicity** |
| **Acid Deposition** | **Pollution to Water: Eutrophication** |
| **Ozone Depletion** | **Minerals Extraction** |
| **Pollution to Air: Human Toxicity** | **Water Extraction** |
| **Pollution to Air: Low Level Ozone Creation** | **Waste Disposal** |
| **Fossil Fuel Depletion and Extraction** | **Pollution to Water: Human Toxicity** |

To aid interpretation, Ecopoints are derived so that the annual environmental impact caused by a typical UK citizen creates 100 Ecopoints. *More Ecopoints indicate higher environmental impact.* The Ecopoints scores have been utilised in the Envest software.

## 4 ENVEST DESIGN TOOL

ENVEST is the first UK software tool for estimating the lifecycle environmental impact of commercial buildings at the building inception stage. The tool provides a holistic approach to the design by:

- Helping to optimise the form of the building, for the least environmental impact over the building life cycle
- Informing choice about the environmental impacts of the main elements of the building structure.
- Providing and maintaining reference data from material manufacturers.
- Aiding designers to balance the environmental impact of the energy and water consumed during the operational life of the building, with the choice of building materials.
- Performing comparisons of various building types.

ENVEST II will be launched in Spring 2002. The original version of ENVEST is for office buildings, and considers the environmental impacts of both the materials used during construction and the energy and resources consumed over the building's life. This is now being extended to allow consideration of schools and hospitals. The most significant change is the addition of WLC results alongside the Ecopoint scores. This is described in more detail below.

Using *minimal* data entered through simple input screens, Envest allows designers instantly to identify those aspects of the building which have the greatest influence on its overall environmental impact and cost.

### 4.1 Building fabric specifications

Envest provides a choice from most mainstream building materials. The range of specifications presented in the tool is based on a sensitivity analysis, which looked at the sensitivity of each element's specification parameters to primary embodied energy. At each stage, a 10% error target was selected as the criterion for inclusion of a parameter or for selecting parameter values needed to represent the range of variation encountered. The principle material-consuming elements are generally upper floors, walls, substructure, external windows and roof elements and to predict embodied energy for whole buildings within 10%, then these elements must be accurate to this degree. For the other elements, achieving the 10% target for the element is less critical because these elements will have a much smaller effect on the result for the building.

Embodied environmental impacts (not only energy) are estimated by calculating the quantity of each material used in individual elements over the life of the building and multiplying it with the Ecopoints per tonne of that material.

### 4.2 Building services specifications

Envest provides a choice from basic building services specifications to enable designers to balance the environmental impact of the energy and water consumed during the operational life of the building, with the choice of building materials. The choices for services include Heating, Lighting, Ventilation, Water, Refrigeration, Office equipment, Lifts and Catering.

Envest calculates the operational environmental impacts of these services by comparing performance against typical performance benchmarks. These benchmarks have been derived from the ECON19 guide, which provides typical and good practice energy and carbon dioxide emission benchmarks for the building as a whole, plus those for individual building services, systems and components.

## 5 ENVEST: UNDERSTANDING YOUR RESULTS.

1 Ecopoint can be described as equal to any (not all) of the following:

- 320 kWh electricity
- 83 $m^3$ Water: enough to fill 1,000 baths
- Landfilling 1.3 tonnes of waste
- Manufacturing 3/4 tonnes brick (250 bricks)
- 1.38 tonnes mineral extraction

Envest provides the facility of measuring impacts per square metre of building gross floor area. Using the tool, it is possible to compare results from a current building to any a user has entered previously. For an architect, it is informative to compare the scores for different buildings designed by your company and as a building owner, it is informative to know the score for your building portfolio, as a complement to the new BRE Environmental Benchmarking for Property Portfolios service.

In addition, it is always useful to compare the result of your building to those designed by others. The following graph provides a range of benchmark figures. The lowest score (i.e. better environmental performance) is for the BRE Environmental Building (1997), the middle score for the Wessex Water HQ design in Bath and the next for the BRE Low Energy Office (1981). All these buildings were designed taking into account environmental impact. A typical building designed without consideration for environmental impacts would be expected to score approximately 40 Ecopoints per square metre.

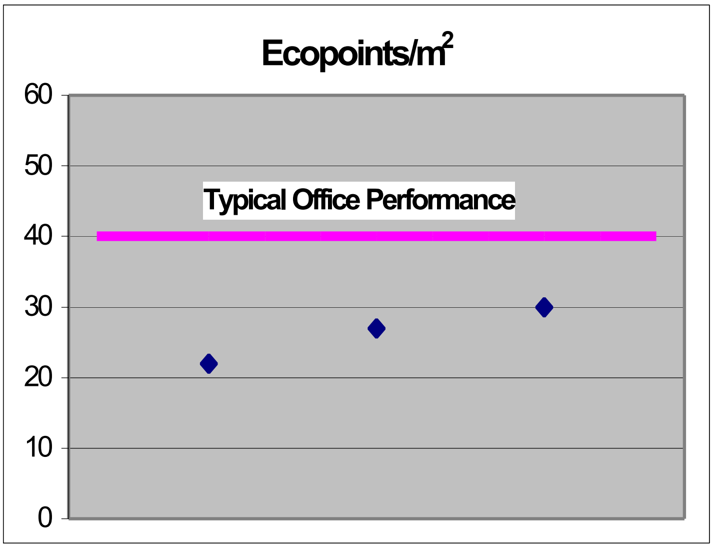


**Figure 2: Benchmarking Environmental Performance in Ecopoints**

## 6 ADDING WLC TO ENVEST

The following activities have taken place to ensure compatibility between WLC and LCA:

1. Mapping of BCIS and UNICLASS onto ENVEST framework.
2. Addition to existing embodied Life Cycle Ecopoints, operational Ecopoints and total Ecopoints of the capital cost, operational cost and total whole life cost as whole building summary. The contribution to Life Cycle Ecopoint and contribution to Whole Life cost indicated against options wherever possible on input sheets.
3. WLC graphs added.
4. Identification of relevant input data and calculation methods to facilitate calculation of additional whole building costs currently not considered - e.g. tax.
5. Ammendment of initial processing databases :

- Materials/Components/Elements/Assets vs Masses of Material
- Materials/Components/Elements/Assets vs Environmental Profiles
- Materials/Components/Elements/Assets vs Replacement lives Hi, Av, Lo
- Materials/Components/Elements/Assets vs Cost Hi, Av, Lo
- Materials/Components/Elements/Assets vs Maintenance mats env-profile
- Materials/Components/Elements/Assets vs Maintenance mats costs Hi, Av, Lo
- Materials/Components/Elements/Assets vs Cleaning mats env-profile
- Materials/Components/Elements/Assets vs Cleaning mats costs Hi, Av, Lo
- Materials/Components/Elements/Assets vs Disposal mats env profile
- Materials/Components/Elements/Assets vs Disposal mats costs Hi, Av, Lo
- Utilities Annual - as per ECON 19 categories parameters for modelling energy
- Utilities Annual - as per ECON 19 categories energy costs
- Utilities Annual - as per ECON 19 categories parameters for modelling water

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- Utilities Annual - as per ECON 19 categories parameters for drainage & effluent
- Environmental Profiles vs weightings vs Ecopoints

## 7 OTHER APPLICATIONS OF ENVEST: TEACHING

Lecturers have responded positively to using Envest with their students as a teaching tool. Of key importance is that they have found it easy to use, enjoyable and a valuable exercise for their students to undertake. Many consider that Envest could usefully be used by students undertaking large-scale projects as part of an environmental analysis of design. This could be both at an early stage in the design, allowing the students to assess the environmental impacts of different design options, and at a later stage, asking students to identify whether they could have reduced the environmental impacts.

Lecturers can provide teams of students with a base building specification and ask them, bearing in mind the normal principles of construction and structures, to try to make the maximum reduction to the overall impact of their building over its life cycle. The basic learning exercise is often based around the following example. All the base buildings had floor areas of 5000 $m^2$ and with the default specifications they had impacts of approximately 35-37 Ecopoints/m2 for operational energy and between 5 and 6 Ecopoints/m2 for Embodied Impacts. . In a trial learning exercise, teams managed to reduce Operational Impacts to between 14 and 17 Ecopoints/m2 and Embodied impacts to between 1.3 and 2.3 Ecopoints/m2, a reduction of just over a half, and two thirds respectively, and an overall reduction of 57%. In going through the exercise, teams were able to explore the full potential of Envest, assessing what was causing the greatest impacts over the life cycle, and by initially targeting these areas, seeing whether significant improvements could be made.

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**Further information:** See www.bre.co.uk/envest