SERVICE LIFE DESIGN FOR THE WESTERN SCHELDT TUNNEL
The western Scheldt tunnel

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Abstract

Due to the high construction costs and the social importance the durability demands for large infrastructures are becoming more and more important. Service life requirements of 100 year or even more are usual. For the bored reinforced concrete tunnel under the Western Scheldt in the Netherlands the requirement was a service life of at least 100 years. No method had been specified to prove this service life. Since the concrete codes are only based on deem-to-satisfy rules for the durability, without any specification to the service life, it was not possible to base the design on existing codes. The service life design has been made on basis of the methodology that has been developed in a research project of the European Community. This project with the name ‘DuraCrete’ has further improved the existing reliability and performance based structural design method by introducing the modelling of degradations and environmental actions. It is believed that the service life design of the Western Scheldt Tunnel is the first project were the DuraCrete approach has been applied in practice.

Keywords: concrete, durability, tunnel, probabilistic design

1 Introduction

Tunnels in the Netherlands are traditionally built by prefabricating large tunnel sections. These sections are shipped to the final location and scuttled. Recently, another construction method is applied: boring tunnels. The main difficulties in boring in soft soils like clay and sand have been overcome. The second bored tunnel in the Netherlands is now under construction. The tunnel consists of two tubes with
an external diameter of 11 m and a length of 6.5 km. The tunnel will cross the Western Scheldt in the south-western part of the Netherlands. The bored tunnel sketched in Figure 1 as a cross-section and in a longitudinal perspective has to be designed against variously induced re-bar corrosion. The bored tunnel is located in chloride contaminated soil and therefore under chloride attack. The inside walls of the tunnel are not ventilated and therefore under the of carbonation. As well, they are exposed to road traffic induced chloride contaminated salt fog and splash environment. Furthermore, leaking joints will eventually lead to chloride attack within the joints and partially at the inside surfaces in particular for deep points of the tunnel.

Fig. 1: Geometry of the bored tunnel under the Western Scheldt

The cross-section of a singular circular segment and the expected environmental loading is presented in the following Figure 2.

Fig. 2: Geometry of the reinforced circular segment of the Western Scheldt bored tunnel and expected environmental loading
In the contract between Rijkswaterstaat (investor) and the Kombinatie Middelplaat Westerschelde (KMW, contractor) a service life of at least 100 year has been fixed, without a specification of the related performances and the reliability level. Schiessl · Raupach · Consulting · Engineering and TNO have been consulted on the service life approach.

2 Historical overview of service life design in practice

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 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 for a large extent empirical. Improving the durability results in increasing building costs without any quantification of the reduction of maintenance costs or failure costs. Current design methods permit only the calculation of whole life cycle costs from assumptions with respect to maintenance and failure rates. Thus, there are 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.

A lack of durability can cause serious safety and serviceability problems for structures. Despite this, at this moment designers have 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 various types of damages and in single cases even collapses can occur with tremendous overall costs for repair.

For large infrastructures the requirements with respect to durability are in general more stringent than the codes. Some examples of storm surge barriers in the Netherlands can demonstrate this. For the Haringvlietsluizen (1965) the requirements were based on the experts opinion of that time. The concrete cover was raised to 70 mm, the water/cement ratio was restricted to 0.45 and blast furnace slag cement was used as binder. For the Eastern Scheldt Barrier (1985) the durability requirements were for the first time expressed in terms of time. A service life of 200 years has been required. For the concrete cover this was not feasible. A mean service life of about 85 years has therefore been accepted. After that period the concrete cover will be replaced. For the Maeslandtkeiring (1995) a service life of 100 years has been specified. The concrete cover of this barrier has been enlarged with a factor $\sqrt{100/50} = \sqrt{2}$. The figure 100 is the required service life and the figure 50 is the service life that is assumed in the Dutch concrete code. In none of these designs has a proper approach for predicting the service life been applied.
3 Design framework

One of the most important reasons that no proper service life designs were made was the lack of a methodology and a design framework. In the past period this has been developed and will be developed further. Some major steps are the CEB Bulletin 238 (Schiessl et al. 1997) and the Brite/Euram project ‘DuraCrete’ that has been finished in February 1999 (TG7 1999). The essential items of the methodology have been copied from the structural design (Siemes and Rostram 1996):

- the performances are related to limit states
- the reference period is similar to the design service life
- the reliability index is used to reduce the failure probability.

The new items in the approach are the introduction of environmental actions and models describing the degradation of the concrete and the reinforcement.

The limit states that have been defined in the structural codes are the serviceability limit state (SLS) and the ultimate limit state (ULS). If a SLS is exceeded the functioning of the structure is restricted.; e.g. the deflection of a beam is more than 20 mm. If an ULS is exceeded the structure has collapsed, fractured, turned over, etc., the static equilibrium has been lost and safety is endangered.

The limit states of the structural codes may be extended with other limit states. For example the watertightness of a tunnel can be expressed as a SLS (for example: the leakage water hinders the traffic in the tunnel) or as an ULS (for example: the leakage water floods the tunnel). Also new types of limit states may be introduced. For example the limit state to reduce the amount of repair. In case of corrosion of the reinforcement it is possible to reduce the amount of repair by requiring that no onset of corrosion may occur.

In Figure 3 an example of a service life distribution is given. The main parameters in such a distribution are the mean value $\mu$ (magnitude parameter) and the standard deviation $\sigma$ (scatter parameter). Instead of the standard deviation we can also use the variation coefficient $V = \sigma/\mu$. By means of the reliability index, $\beta$, the probability that the service life is lower than the design service life $L_1$ is identified.

Fig. 3: Example of a probability density function for the service life
With every performance an unique distribution function is connected, depending on the geometry of the structure, material properties, mechanical loads and the environmental actions. This implies that for every relevant performance the distribution function has to be established, the design service life as well as the reliability index must be defined. In this respect it is important to define a limit for the performance that separates the adverse state form the required state.

In Table 1 some values for the reliability index, $\beta$, and corresponding approximate failure probabilities are given (European Code EC1 and Dutch Code NEN 6700). These reliability indexes can be used if lack of durability leads to an event that leads to an unacceptably loss of serviceability or to a loss of structural safety (Siemes and Rostam 1996). If failure leads only to economical consequences serviceability limit states (SLS) are applied, e.g. onset of corrosion. If failure leads to severe consequences, e.g. (total loss of the structure or victims) ultimate limit states (ULS) are applied (collapse due to overloading or due to excessive material degradation). Other intermediate stages may be defined by the owner (investor) in order to reduce repair costs, ensuring the structural appearance, etc..

Table 1: Examples of some reliability indexes in structural codes

<table>
<thead>
<tr>
<th>Type of Performance</th>
<th>Reliability Index in a Period of 50 Year</th>
<th>Approximate Failure Probability in 50 Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td>3.8</td>
<td>3.6</td>
</tr>
<tr>
<td>SLS</td>
<td>1.5</td>
<td>1.8</td>
</tr>
</tbody>
</table>

4 Design calculations

4.1 Introduction

In this paper we will restrict to the design of the tunnel lining, although in practice the whole tunnel has been based on a service life design. The following limit states were chosen:

- Onset of corrosion
- Corrosion induced spalling and corresponding loss of watertightness
- Collapse of the structure.

Onset of corrosion is a serviceability limit state. When corrosion has started, maintenance is much more complicated to perform and therefore much more expensive than for a structure without initial corrosion activity. Consequently there is an economic risk present. Corrosion induced spalling in the area of joints can lead to a failure in watertightness. In dependency to the failure probability the operating costs will increase (pumps, etc.) to an upper limit. The highest economic damage at the serviceability level is reached when the leakage rates are so high that the tunnel can not be operated. The collapse of the structure should be avoided at a highest
reliability level (ultimate limit state, ULS) not only to avoid with corresponding low
damage, but also to safe human life.

Finally, the following set of operational limit states and their related reliability
indexes was derived (Table 2).

Table 2: Set of operational limit states

<table>
<thead>
<tr>
<th>Event</th>
<th>Limit State</th>
<th>Reliability Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Onset of corrosion</td>
<td>SLS and repairable</td>
<td>1.5 - 1.8 EC1 – NEN 6700</td>
</tr>
<tr>
<td>Corrosion induced spalling and corresponding failure in watertightness</td>
<td>SLS and repairable</td>
<td>2.0 - 3.0 Proposal</td>
</tr>
<tr>
<td>Collapse of the structure</td>
<td>ULS</td>
<td>3.6 - 3.8 NEN 6700 - EC 1</td>
</tr>
</tbody>
</table>

4.2 Design example: Concrete cover within the joints

Due to structural design constraints the concrete cover within the joints should
be minimised. Therefore, the joint with its restricted cover of $x_c$ (joint) was identified as
the most critical detail with regard to durability. In the case of permanent leakage, the
cement is subjected to chlorides coming from the chloride contaminated soil. Constantly humid conditions are assumed; carbonation can be neglected. The
identified deterioration mechanism is in this case the chloride induced corrosion. The
question that has to be answered here as an example is: Which material resistance
against chloride penetration in combination with which concrete cover $x_c$ is necessary
in order to fulfil the investors requirements? In the following paragraph the carried
out design of the concrete cover $x_c$ (joint) is presented for a serviceability limit state
(Onset of corrosion: $T_{Service\ Life} = 100$ year with a minimum reliability index of
$\beta_{0,SLS} \geq 1.50 - 1.80$).

4.3 Deterioration model

In the case of a chloride induced reinforcement corrosion, deterioration models
are required, describing the initiation period as well as the propagation period. As
mentioned before, here we will only discuss the initiation period.

The following deterioration model is describing the time dependent process of
chloride penetration until the depassivation of the re-bar will occur. The suggested
model to predict the duration of the initiation period in the case of chloride induced
re-bar corrosion has been identified within an intensive literature research carried out
in a Brite/EuRam-project DuraCrete (Alisa et al. 1998), Equations (1), (2) and (3)):

\[
x(t) = 2 \cdot C(Cr) \cdot \sqrt{k_t \cdot D_{RCM,0} \cdot k_e \cdot k_c \cdot \left(\frac{t_0}{t}\right)^n} \cdot t
\]

with:

\[
k_t \cdot D_{RCM,0} = D_0
\]
\[
C_{(Crit)} = \text{erf}^{-1}\left(1 - \frac{C_{Crit}}{C_{SN}}\right) 
\]

(3)

with:
- \(x_c\): concrete cover in [mm]
- \(D_0\): effective chloride diffusion coefficient at defined compaction, curing and environmental conditions, measured at time \(t_0\) in \([m^2/s]\)
- \(D_{\text{RCM},0}\): chloride migration coefficient at defined compaction, curing and environmental conditions, measured at time \(t_0\) in \([m^2/s]\)
- \(C_{\text{Crit}}\): chloride threshold level in [wt.-%/Cl/binder]
- \(n\): factor which takes the influence of age on measured material property into account
- \(k_t\): constant parameter which transfers the measured chloride migration coefficient \(D_{\text{RCM},0}\) into a chloride diffusion coefficient \(D_0\)
- \(k_e\): constant parameter which considers the influence of environment on \(D_0\)
- \(k_c\): constant parameter which considers the influence of curing on \(D_0\)
- \(\text{erf}^{-1}\): inverse of the error function
- \(C_{SN}\): surface chloride level in [wt.-%/Cl/binder]
- \(t\): exposure period in [year], \(t_0\): reference period in [year], in this case \(t_0 = 28\) day

4.4 Data

The data for the service life design performed for the tunnel segments are given in the following Table 3 (the required variables under consideration can be drawn out of Equation (1) and (3)):

<table>
<thead>
<tr>
<th>Variable No</th>
<th>Parameter</th>
<th>Dimension</th>
<th>(\mu)</th>
<th>(\sigma)</th>
<th>Distr. Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(x_c) – Concrete Cover</td>
<td>[mm]</td>
<td>37</td>
<td>2</td>
<td>Expon. Distr.</td>
</tr>
<tr>
<td>2</td>
<td>(D_{\text{RCM},0}) - Cl-Migration Coef.</td>
<td>([10^{-12}m^2/s])</td>
<td>4.75</td>
<td>0.71</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>3</td>
<td>(C_{\text{Crit}}) – Critical Chloride Content</td>
<td>[wt.-%/binder]</td>
<td>0.70</td>
<td>0.10</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>4</td>
<td>(n) - Age Exponent</td>
<td>[-]</td>
<td>0.60</td>
<td>0.07</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>5</td>
<td>(k_t) - Factor Test</td>
<td>[-]</td>
<td>0.85</td>
<td>0.20</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>6</td>
<td>(k_e) - Factor Environment</td>
<td>[-]</td>
<td>1.00</td>
<td>0.10</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>7</td>
<td>(k_c) - Factor Execution</td>
<td>[-]</td>
<td>1.00</td>
<td>0.10</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>8</td>
<td>(C_{SN}) – c(Cl) - Concrete Surface</td>
<td>[wt.-%/binder]</td>
<td>4.00</td>
<td>0.50</td>
<td>Normal Distr.</td>
</tr>
<tr>
<td>9</td>
<td>(t_0) – Reference Time</td>
<td>[year]</td>
<td>0.0767</td>
<td>-</td>
<td>Deterministic</td>
</tr>
</tbody>
</table>
In the following it will be shown for the first three listed stochastic variables how each distribution function was obtained.

4.4.1 Concrete cover

During the production of the circular segments it is ensured that the concrete cover will be built in very precisely. It is supposed that reinforcement can be embedded in a $x_c \geq 35\,\text{mm}$ concrete cover, assuming precise fabrication and attention to quality assurance measures. The geometrical variable concrete cover is therefore introduced as an exponential distribution with a relatively low standard deviation.

4.4.2 Chloride migration coefficient: $D_{\text{RCM,0}}$

For design purposes an important starting measure is the material resistance when focusing on the expected deterioration processes. Suitable results may occasionally be drawn out of the literature, which might be used as starting parameters in a service life design calculation. Oftentimes one can not draw quantitative results out of the literature, when working with special concrete mixes with for example very low water binder ratios and high contents of plasticiser. Therefore, it is essential to determine the efficiency of the materials in use by testing them in basic tests. This is to identify the suitability of the designed concrete mix. In this context the decisive material resistance is the chloride diffusion coefficient.

The determination of chloride diffusion coefficients is a rather complex and time-consuming procedure (Gehlen and Ludwig 1998). In practice, chloride penetration into concrete structures follows a non-steady state process. For this process, conventional methods are to immerse the concrete specimens in chloride contaminated solutions with constant chloride concentration. After immersion, the chloride profile is measured by sampling the specimen successively from the exposed surface and by analysing the total chloride content in each sample. The chloride diffusion coefficient can be found from the Fick’s second law by curve fitting. In order to prepare a meso-level durability design in time before the structural elements are constructed, the measured material performance should be known as soon as possible. Among different rapid test methods, the rapid chloride migration method (RCM) revealed to be theoretically the clearest, experimentally the most simple and related to precision (repeatability) the most promising tool (Gehlen and Ludwig 1998). Diffusion data measured with conventional immersion methods correspond well to data determined with the rapid chloride migration method (Gehlen and Ludwig 1998; Tang 1996). How the measurement has to be undertaken can be seen in Gehlen and Ludwig (1998) and Tang (1996). The material under investigation gave values as provided in Table 3 as mean and standard deviation.

In addition to the measured performance at the time $t_0$ one has to consider the sometimes substantial difference in the time dependent development of the material resistance (Siemes et al. 1999; Bamforth 1998). The increase of the material resistance, or expressed by the opposite, the reduction of the chloride diffusion coefficient, will be expressed by the age exponent $n$ (Variable 4 of Table 3). Typical quantities of $n$ are given in Siemes et al. (1999) and Bamforth (1998).
4.4.3 Critical corrosion inducing chloride content: $C_{\text{crit}}$

In the Design Guide of the Comité Euro-International du Béton (CEB) for durable reinforced concrete structures of 1989 (CEB 1989), the relationship between the critical chloride content depending on the moisture content of concrete and on the quality of the concrete cover has been illustrated (Figure 4).

Whereas for permanent dry or permanent water saturated concrete practically no re-bar corrosion is possible, and therefore higher values for the critical chloride content can be admitted, appears the lowest value for the critical chloride content under permanent moist or alternating environmental conditions. In the case of a large concrete cover and a good concrete quality, which will be basically obtained by a low water/binder ratio and a suitable curing of the concrete, as well as through the use of additives, one can achieve higher critical chloride contents as compared to a insufficient quality of the concrete cover. The lowest critical threshold value for the chloride content was determined by Breit (1997) for various mortar mixtures. The surveys of Breit allow for the first time to introduce the critical corrosion inducing chloride content as a stochastic variable. An adapting test for the obtained results has shown that a normal distribution ($\mu = 0.48$, $\sigma = 0.15$) describes the results in a sufficiently exact manner.

In the surveys of Breit a set of different mixtures was investigated. The concrete cover had a value of only $x_c = 7.5$ mm. Making use of only one mixture and taking into account higher concrete covers, one can therefore estimate higher averages (due to increased concrete cover) and lower standard deviations (only one mixture). All the explained relationships justify the above mentioned assumption.
4.5 Result
With the arranged data it is now possible to calculate whether the stochastic variable \( x_c \), which was applied in the design (Table 3) is sufficient to proof a serviceability limit state related minimum service life of \( T = 100 \) years. The required statistical calculation was carried out with the software package STRUREL (RCP GmbH: STRUREL 1995). The result of the evaluation is shown in Figure 5.

![Reliability Index vs Time of Exposure](image)

**Fig. 5: Reliability index versus time of exposure (SLS: onset of corrosion)**

Figure 5 shows, that with the tested material and the planned concrete cover one can achieve a durable structure concerning chloride induced steel corrosion with the minimum required safety according to the fixed serviceability limit state ‘Onset of corrosion’. The obtained reliability index is \( \beta_{SLS} = 1.50 \) \((p_f = 0.068)\), in accordance to the minimum required value of Table 2 \((\beta_{0,SLS} = 1.50)\).

5 Additional measures

Beyond the basic testing (laboratory test), the quality level achieved in the laboratory should be reached on the construction site as well, since the quality of the built-in material decides over the service life of the construction. Criteria for the final examination (acceptance criteria) of the tested material resistances can be determined following EC1, appendix D. The continuous examination of the material variable \( D_{RCM,0} \) cannot be undertaken on the construction site without significant effort, because for the construction site this method is too complicated to perform. Extensive surveys at the Institute for Material Research at the Technical University of Aachen (ibac) have shown a good correlation between the chloride migration coefficient \( D_{RCM,0} \) and the electrolytic resistivity of concrete \( \rho_{WER,0} \) (Figure 6)
untaken with the so-called Wenner-probe. How to execute the Wenner method is
described in detail in Gehlen and Ludwig (1998). Considering this aspect, the
material resistance towards a chloride penetration was verified for the tunnel
segments on concrete cubes stored submerged on the construction site by an indirect
examination of electrolytic resistivity of the concrete.

![Graph showing the relationship between $D_{RCM,0}$ and $\rho_{WER,0}$](image)

**Fig. 6: Relationship between $D_{RCM,0}$ and $\rho_{WER,0}$**

Further it has to be assured by a permanent measurement of the concrete cover,
that the measured performance with regard to $x_c$ is equal to the required performance
$x_c$. Finally, other additional measures like hydrophobation in areas of restricted
covers can be considered. By consideration of partly applied hydrophobation within
the joints, the re-calculated reliability index at time $t = 100$ years is $\beta_{SLS} = 2.25$
($p_f = 0.012$), higher than the required value of $\beta_{0,SLS} = 1.50 - 1.80$.

6 Concluding remarks

In the last decade much effort has been spent on the service life design of
structures and especially of concrete structures. This has finally led to a situation
where it was possible to make a fully performance and reliability based service life
design for a real structure: the Western Scheldt Tunnel.

During the design, simplifications have been made. For example the target
reliability indexes have directly been adopted from the structural design, both for the
SLS and the ULS. For the ULS this is more or less obvious, as exceeding this limit
state will lead both for the structural design and the service life design to the same situation, namely loss of the structure. For some of the SLS’s this is less clear. For example the onset of corrosion or the corrosion induced spalling are limit states that are related to the risk for repairs. This is primarily an economic issue and an economic optimisation of the reliability index is obvious. The optimisation is hardly possible because the amount of repair and the locations for the repair are difficult to estimate.

The service life design has been based on the reliability during the design service life. In other words it has been based on the total failure probability during the service life. Due to the degradation we have a situation where the failure probability grows with time. The failure rate (failure probability per year) in the first years is therefore low and in the last years relatively high. An improvement can be made by also restricting the failure rate.

The service life design has been made for the whole tunnel lining without differentiating the exposure zones. Especially for long tunnels with a high number of similar concrete elements this may lead to the acceptance that a high number of elements will fail. To restrict this the exposure zones can be differentiated or the target reliability index can be related to the amount of possible damage. For a small amount of failing elements a low reliability index can be applied. For a high amount of failing elements a high reliability index should be applied.

7 References


