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; ρ_c and ρ_a are the mass densities of cement and aggregates respectively; D_{H2O} is the diffusion coefficient in an infinite solution (=1.6×10^{-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 0.1μA/cm^2, 1μA/cm^2 and 10μA/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

$$i_{corr}(t_p) = i_{corr}(1) \times 0.85 \times t_p^{-0.29} \quad t_p \geq 1\ year \tag{3}$$

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.

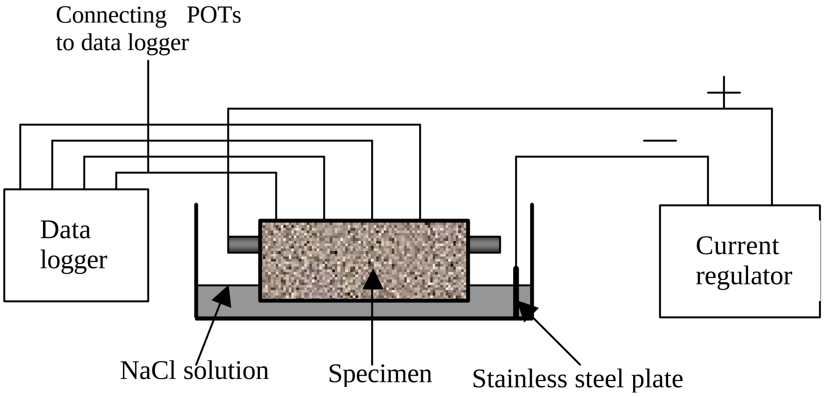

Figure 1. Experimental Set up of Accelerated Corrosion Test

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.

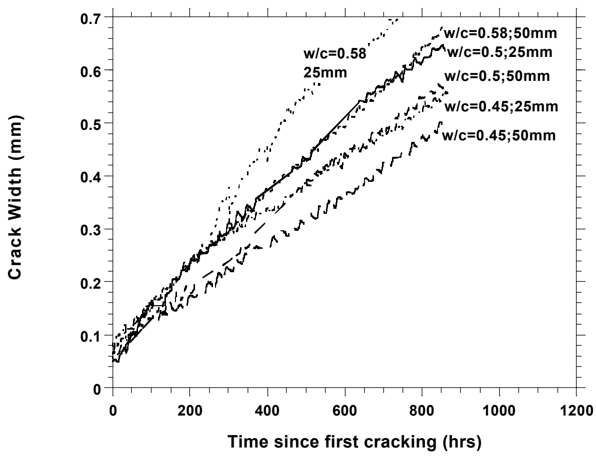

Figure 2. Crack Propagation Since First Cracking

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

$$t_{ser} = \mathrm{A} \times 10^{-3} \times \left(\mathrm{wc/C}\right)^{-\mathrm{B}} \qquad (4)$$

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.

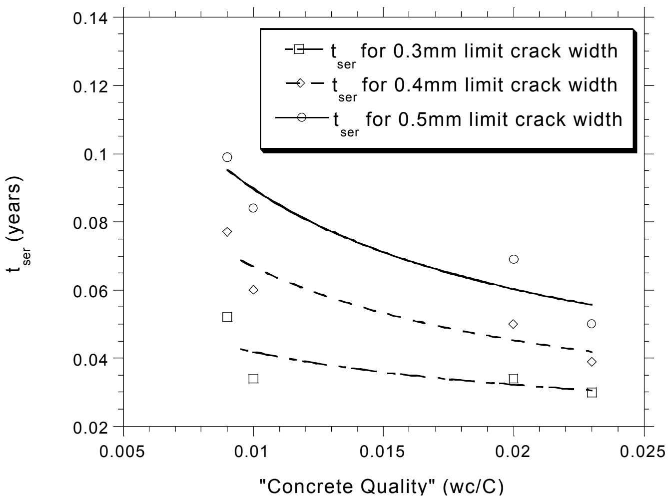

Figure 3. Effect of Concrete w/c Ratio and Cover on Crack Propagation

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.

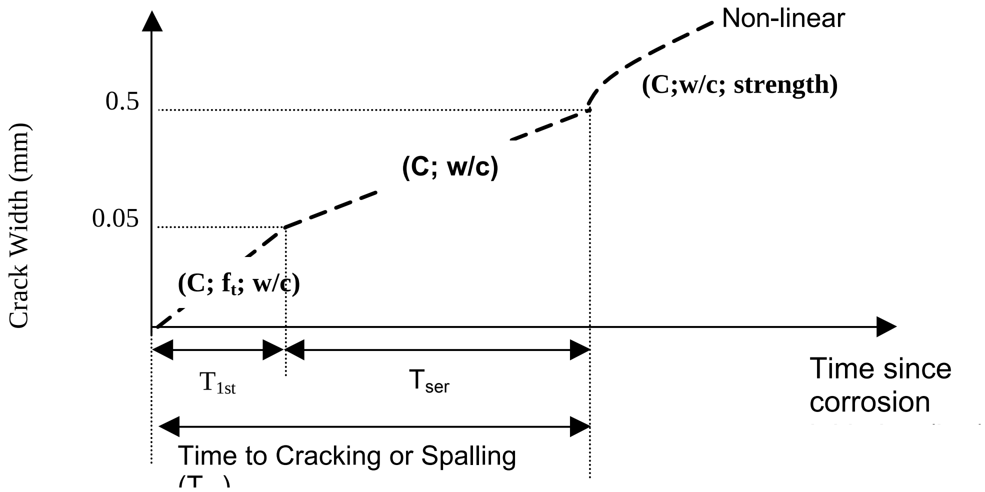

Figure 4. Modelling Time Dependent Crack Growth

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

$$t_{ser} = \left[A \times 10^{-3} \times \left(wc / C\right)^{-B}\right] \times \frac{100}{\mathrm{i}_{corr}(1)} \tag{5}$$

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

$$X_i(t) = \frac{1}{T}\int_{t-T/2}^{t+T/2} X(t)\,dt \tag{8}$$

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

$$Var[X_i] \equiv \sigma_i^2 = \gamma(T)\sigma^2 \tag{9}$$

where $\gamma(T)$ is the variance function of $X_i(t)$ and σ_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

$$COV\lfloor X_i, X_j \rfloor = (\sigma_i \sigma_j) \times \rho_{X_i, X_j} \tag{10}$$

where $\rho_{Xi,Xj}$ is the coefficient of correlation between the local averages X_i and X_j and is expressed linearly as\

$$\rho_{X_i,X_j} = \frac{1}{2}\left[T_0^2\gamma(T_0) - T_1^2\gamma(T_1) + T_2^2\gamma(T_2) - T_3^2\gamma(T_3)\right] \tag{11}$$

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 σ^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 θ, where θ is the scale of fluctuation. It is assumed herein that θ = 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	σ = 16.5mm	Normal
f'_{cyl}	f'_c+7.5MPa	σ = 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).

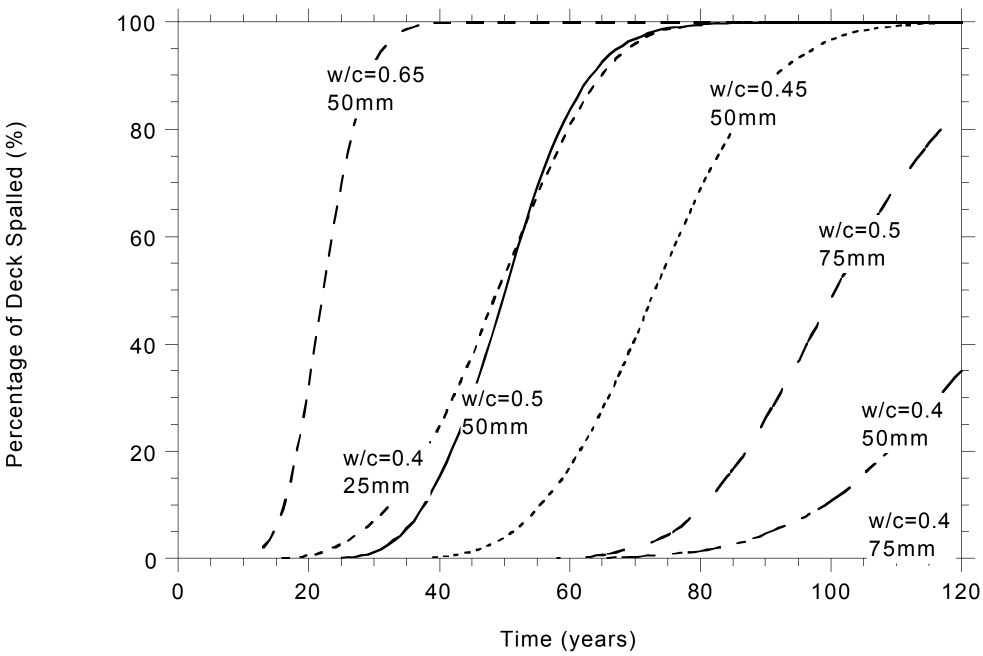

Figure 5. Effects of Concrete Cover and w/c Ratio on Concrete Cracking and Spalling for Sea-spray

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.

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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.

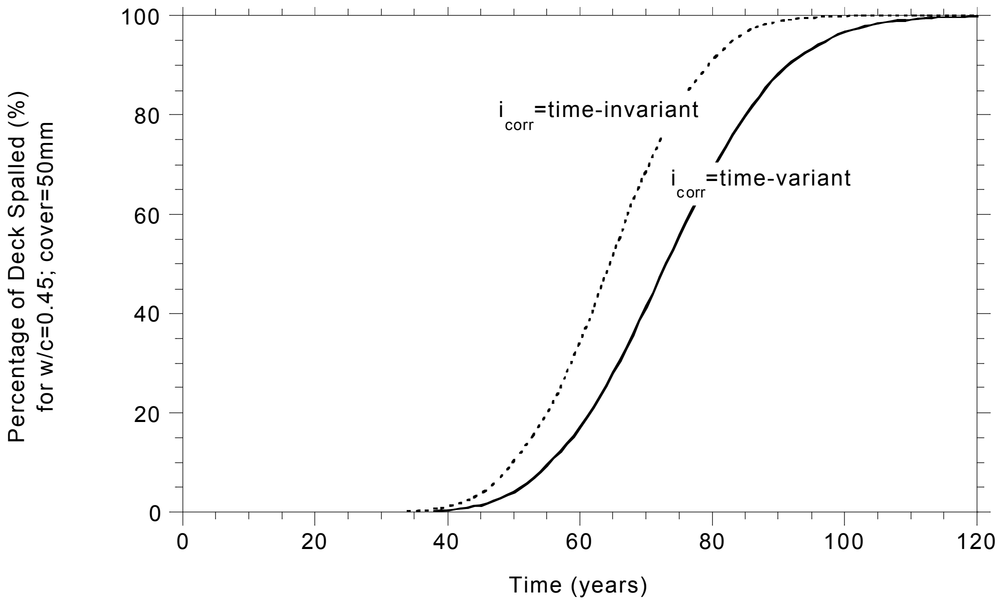

Figure 6. Effect of Modelling Corrosion Rate as a Time-Invariant Variable, for 50mm cover and w/c = 0.45.

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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