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Reliability-Based Prediction of Chloride Ingress and Reinforcement Corrosion of Aging Concrete Bridge Decks - A Case Study Investigation

Zoubir Lounis[1] and Lamya Amleh[2]

Abstract: *This paper presents a probabilistic approach for predicting the chloride contamination of concrete and reinforcement corrosion, which takes into account the uncertainty associated with the analytical models of chloride transport, corrosion initiation, and damage accumulation, material properties, structural dimensions, and applied environmental and mechanical loads. The proposed approach is illustrated on an aging reinforced concrete bridge deck that has been exposed to chlorides from deicing salts for forty years. An extensive non-destructive and destructive evaluation of the corrosion-damaged deck was undertaken. The field survey data showed a considerable level of variability in all parameters measured with coefficients of variation ranging from 34% for the concrete cover depth to 86% for the diffusion coefficient. The distributions of the chloride concentration at the level of the top reinforcement mat and the time for its corrosion initiation were generated using Monte Carlo Simulation. The simulated results were very close to the field data, which illustrates the prediction capabilities of probabilistic methods as opposed to deterministic methods, as well as to some non-destructive evaluation methods.*

Introduction

The deterioration of concrete structures due to chloride-induced reinforcement corrosion is recognized as one of the major challenges facing the owners of facilities and infrastructure systems made up of concrete structures. Despite their durability when compared to steel and timber structures, reinforced and prestressed concrete structures are vulnerable to the damaging effects of corrosion induced primarily by chlorides (from deicing slats and seawater) and to a lesser extent to carbonation-induced corrosion. It is estimated that one-third to one-half of the projected bridge rehabilitation costs in North America are related to bridge deck deterioration (Cady and Weyers 1983; Lounis and Mirza 2001). The corrosion of the reinforcement leads to concrete fracture through cracking, delamination and spalling of the concrete cover, reduction of concrete and reinforcement cross sections, loss of bond between the reinforcement and concrete, reduction in strength (flexural, shear, etc.), and ductility. As a result, the safety and serviceability of concrete structures are reduced, and their useful service lives shortened.

Depending on the importance of the structure and the consequences of its failure, different rehabilitation options may be implemented to upgrade the structure in order to ensure its safety, serviceability and functionality using a life cycle cost analysis.

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The prediction of the performance of concrete structures subjected to chlorides requires a thorough understanding and reliable modeling of several complex mechanisms, which include: (i) transport of chlorides into concrete; (ii) corrosion initiation through depassivation of the steel and destruction of the protective film layer; (iii) damage initiation and accumulation (cracking, delamination, and spalling); and (iv) impact of corrosion on bond and strength. The problem is further complicated by considering the combined effects of stresses and damage-induced by corrosion and mechanical loads. This is critical for a reliable prediction of the safety and serviceability of concrete structures and their remaining life.

The prediction of the service life of concrete structures built in aggressive environments further complicated by the considerable uncertainty in the damage initiation and accumulation models under the combined effects of applied loads and environmental agents. Furthermore, uncertainties arise from the variability of material properties, structural dimensions, traffic loading, environmental conditions, and failure criteria. Therefore, a probabilistic modeling of the performance and service life of concrete structures is required at both design and operation stages. The modeling of the uncertainty leads to reliable service life prediction models that provide mean values, extent and type of scatter. These models can be developed by using the data obtained from long term in-service exposure, accelerated laboratory testing, or from the use of fundamental mechanistic models (Lounis and Madanat 2002).

In this paper, probabilistic approaches for predicting the service life of concrete structures with emphasis on predicting the chloride concentration at steel level and the corrosion initiation time are presented with an illustration on a concrete bridge deck that was exposed to chlorides from deicing salts for forty years. The proposed approach provides a rational modeling of all uncertainties associated with the processes of chloride transport, corrosion initiation, and corrosion propagation.

Service Life of Concrete Structures in Chloride-Laden Environments

The performance and service life of concrete structures exposed to chlorides can be idealized by the modified version of Tuutti's two-stage model (Tuutti 1982) shown in Fig.1, namely:

- (i) *Initiation stage*- in which chlorides penetrate the concrete cover, reach the first layer of reinforcing steel, and accumulate over time until they reach the so-called "threshold level" to initiate corrosion. The corrosion initiation time is dependent on several parameters, including the concrete cover depth, surface chloride concentration (function of amount of deicing salts used), quality of the concrete cover (permeability, diffusivity), type of reinforcing steel (black steel, epoxy-coated black steel, stainless steel, galvanized steel, etc.), and the chloride threshold level for corrosion initiation.
- (ii) *Propagation stage*- in which the stresses induced by the expansion of corrosion products lead to fracture of concrete (cracking, delamination, spalling) , loss of ultimate strength, loss of bond between steel and concrete and ultimately

failure. It is primarily dependent on the rate of corrosion, fracture properties of concrete, reinforcement area, size and spacing.

The definition of failure or service life corrosion-damaged concrete structures is not a straightforward task. Service life can be defined as the time at which any of the following limit states are reached: onset of corrosion, cracking, delamination, spalling, or accumulated damage reaching some specified amount. An appropriate definition of failure, and consequently service life should consider the acceptable risk of failure, which depends on the risk of loss of life and injury, type of structure, mode of failure, etc. Figure 1 illustrates the different possible values of the propagation time and service life.

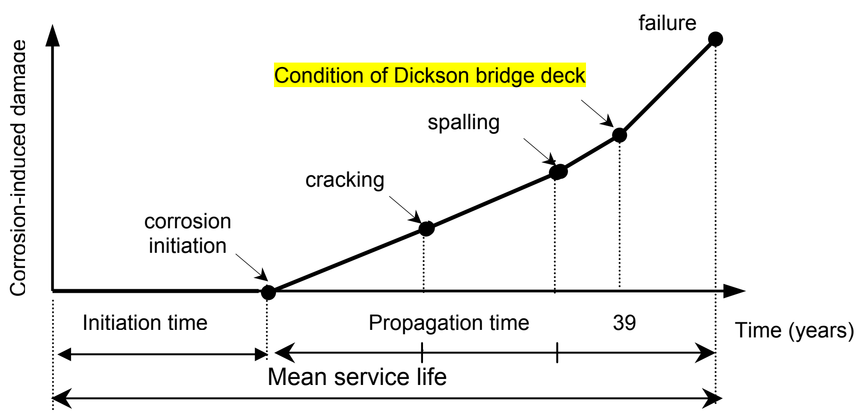

Figure 1. Service life of reinforced concrete structures exposed to chlorides

Modeling the Uncertainty in Service Life of Concrete Bridge Decks

In addition to being exposed to corrosion-induced damage, concrete bridge decks are also subjected to the effects of mechanical loads (dead loads, superimposed dead loads, and traffic loads), which cause damage (vertical and/or diagonal cracks, spalling, deformations), as well as partial or total collapse due to an overload, or/and understrength. The prediction of the safety and serviceability of existing concrete bridge decks and the planning of their maintenance needs are very complex problems. This is due to the multitude of causes of deterioration and failure mechanisms and their interaction, which are very hard to quantify. The main causes of failure may include aggressive environments; overstress due to heavy traffic load; accidental impacts; inadequate design, protection, and construction; aging; and insufficient inspection and maintenance. Both the external effects, and the material and structural parameters are time-dependent and random in nature, with considerable levels of uncertainty. As a result the response of the structure is also random with large fluctuations from the mean value identified by high coefficients of variation or low signal-to-noise ratios. This requires the use of stochastic deterioration models to predict the structural response. The sources of uncertainty can be classified as: physical uncertainty; statistical uncertainty; model uncertainty; and decision uncertainty (Melchers 1987).

The physical uncertainty is that identified with the inherent random nature of the basic variables such as: (i) variability of structure geometry (e.g. concrete cover depth); (ii) variability of material properties (strength, diffusivity, etc.); (iii) variability of micro-environment (e.g. surface chloride concentration); (iv) variability of applied loads; and (v) variability of initial condition.

The statistical uncertainty arises from modeling the parameters and / or performance indicators using simplified stochastic processes or random variables by using lower order of stochastic correlation of stochastic processes or assuming independence of random variables. It also arises from using a limited sample size to estimate the statistical parameters or probabilistic models of the model parameters.

The model uncertainty results from the use of simplified physical models to describe the damage initiation or damage growth mechanisms, such as chloride transport, corrosion initiation, cracking, spalling, collapse, etc. This modeling uncertainty includes: (i) use of a simplified diffusion law to model the chloride transport mechanism; (ii) use of simplified chloride threshold level to define the corrosion resistance of concrete structures; and (iii) use of a simplified resistance degradation model in the propagation stage to assess the safety and serviceability of the structure.

The decision uncertainty is that associated with the definition of the acceptable level of damage or limit state or acceptable probability of failure for both serviceability and ultimate limit states, as mentioned earlier.

Therefore, given the considerable uncertainty that affects the material, structure, environment, and loading, and the resulting structural response, a probabilistic modeling of concrete bridge deck deterioration is an imperative to achieve reliable predictions. This probabilistic modeling will provide a higher level of reliability and practicality when compared to deterministic modeling (Ditlevsen 1984; Melchers 1987; Mori and Ellingwood 1993; Frangopol et al 1997; Lounis and Siemes 1998; Stewart and Rosowsky 1998; Lounis and Mirza 2001; Lounis and Madanat 2002). Ditlevsen (1984) stated: *"Probabilistic models are almost always superior to deterministic models of equal level of complexity in the sense that the former have considerable higher threshold of realism when dealing with phenomena taking place in uncertain environments"*.

Probabilistic Modeling of Chloride Ingress into Concrete

A reliable prediction model for the ingress of chlorides into concrete should consider the complex combination of several transport processes (Neville 1995; Kropp et al. 1995) that include diffusion, capillary sorption (i.e. absorption of water containing chlorides into unsaturated concrete), and permeation (i.e. water flow in concrete due to a pressure gradient). Once the chlorides have penetrated the concrete cover and reached the reinforcement and their concentration is above the so-called "threshold level", the corrosion of the reinforcement can be initiated. However, for the prediction of the ingress of chlorides into a concrete bridge deck exposed to the periodic application of deicing salts, diffusion can be assumed to be the governing transport mechanism (Cady & Weyers 1983). Therefore, the time-dependent distribution of chloride concentration over the depth of the bridge deck can be obtained from the solution of Fick's second law of diffusion assuming that the concrete deck as a homogeneous isotropic semi-infinite medium. Assuming the initial condition C(x,t=0)=0; boundary condition C(x=0,t) = C_s (constant) and constant diffusion coefficient, the chloride content at depth x and time t is given by:

$$C(x,t) = C_s[1 - \mathrm{erf}(\frac{x}{2\sqrt{Dt}})] \tag{1}$$

where C_s is the chloride concentration at the exposed surface (assumed constant), and D is the diffusion coefficient of chlorides into concrete.

Despite its simplicity and extensive use, this model has some shortcomings, because: (i) the diffusion coefficient is not a constant but rather depends on time, temperature, and depth because of the heterogeneous nature and aging of concrete (Cady and Weyers 1982; Neville 1995; Kropp et al. 1995); and (ii) the top surface of the bridge deck is subjected to a continually changing chloride exposure. In general, the values of the surface chloride concentration and diffusion coefficient can be estimated from Eq. (1) by determining the best fit curve through field data obtained from chloride profiles at different exposure times. The chloride concentration at the deck surface varies with the season, however at some shallow depth near the deck surface (within the first 10mm), the chloride concentration, referred to as near-surface chloride concentration can be assumed at a quasi-constant maximum (C_s) value (Cady & Weyers 1982).

In this paper, the uncertainties associated with the transport model and its parameters are considered by using the "apparent chloride diffusion coefficient" concept and modeling the surface chloride concentration and diffusion coefficient as random variables with probability density functions $f_{Cs}(c)$ and $f_D(d)$, respectively, which are obtained from the field measurements of the chloride profiles. The apparent chloride diffusion coefficient is determined by fitting the solution of Fick's second law of diffusion to measured chloride profiles expressed in terms of total chloride concentrations (including both free and bound chlorides). Since only the chlorides dissolved in the pore solution (free chlorides) are responsible for the initiation of the corrosion process (Tuutti 1982; Neville 1995; Kropp et al. 1995), this procedure yields only the value of the so-called apparent diffusion coefficient because chloride binding is not taken into account. The probabilistic distribution of chloride concentration at a given depth and exposure time is generated using Monte Carlo simulation.

Probabilistic Modeling of Corrosion Initiation Time

The corrosion initiation time (T_i) depends on the rate of ingress of chlorides into concrete, surface chloride concentration, depth of concrete cover, and the value of the threshold chloride level. The corrosion is assumed to start when the chloride concentration at the reinforcement level reaches the "threshold level". Using Eq. (1) and assuming the same initial and boundary conditions, the time to onset of corrosion is determined as follows:

$$T_i = \frac{d_c^2 [\mathrm{erf}^{-1}(1 - \frac{C_{th}}{C_s})]^{-2}}{4D} \tag{2}$$

where d_c: depth of concrete cover; D: chloride diffusion coefficient; C_{th}: threshold level of chloride concentration; C_s: surface chloride concentration.

As mentioned earlier, given the uncertainties in the concrete cover depth, surface chloride concentration, chloride diffusion coefficient and threshold chloride level, an accurate prediction of the service life is not possible using a deterministic model. If failure is defined as the onset of corrosion, then the probability of corrosion at time t, is given by:

$$P_f(t) = P(T_i \leq t) = \int_0^t f_{Ti}(x)dx \tag{3}$$

where $f_{Ti}(x)$ is the probability density function of time to corrosion initiation, which is generated by Monte Carlo simulation using the probability density functions of D, d_c, C_s obtained from field measurements, as well as C_{th} . There is no consensus regarding the determination of the threshold chloride level (Tuutti 1982; Neville 1995; Kropp et al. 1995).

Probabilistic Modeling of Corrosion Propagation Time

The propagation time is defined as the time from the onset of corrosion until such time when a critical damage level or 'limit state' is reached. These limit states include the serviceability (excessive cracking, delamination, spalling, deformation), and ultimate limit states (e.g. flexural failure, shear failure, punching shear failure). A reliability-based assessment of the propagation period and total service life is possible through the modeling of both the load and resistance as stochastic processes. The resulting problem can be solved using time-dependent reliability analysis methods (Melchers 1987; Mori & Ellingwood 1993; Frangopol et al. 1997; Stewart and Rosowsky 1998; Lounis et al. 1998, 2001, 2002). If the resistance is assumed to decrease with time according to some degradation function $\alpha(t)$, i.e. $R(t)= \alpha(t)R_o$, which is assumed independent of the loading history (as is the case of deterioration due to environmental factors) but dependent on the rate of corrosion of the reinforcing steel, the probability of failure can then be calculated as follows (Mori & Ellingwood 1993; Frangopol et al. 1997) :

$$P_f(T_L) = 1 - \int_0^\infty \exp\,[-\nu\{T_L - \int_0^{T_L} F_S[\alpha(t)R_o]dt\}]\, f_{Ro}(r)dr \tag{4}$$

where $P_f(T_L)$: probability of failure within the time interval $[0\text{-}T_L]$; T_L: service life; $\alpha(t)$: resistance degradation function; R_o: initial resistance; f_{Ro}: probability density function of the initial resistance; F_s: cumulative distribution of the load; ν: rate of occurrence of loading, P_{fmax}: maximum acceptable probability of failure in the time interval $[0\text{-}T_L]$.

It should be pointed that the collapse mode of concrete bridge decks supported by girders is generally governed by the punching shear failure mode as opposed to the bending failure mode. This is due to the effects of the in-plane compressive forces, referred to as compressive membrane action. This action enhances the ultimate capacity of bridge decks, which is found to be well above the assumed flexural strength. This action modifies the behavior of decks on girders, which tend to act as arches between supporting girders.

Application to the Dickson Bridge Deck

The proposed approach is applied to the Dickson bridge in Montreal, Canada, which was demolished in 1999. The bridge had a total length of 366 m, width of 27 m and was built in 1959 at a cost of $1M dollars. The superstructure consisted of reinforced concrete T-girders in the end spans and concrete deck on steel girders in the central spans. The bridge deck had a specified design thickness of 150 mm, 25 mm concrete cover depth for top reinforcement mat, and 28 MPa concrete strength. Prior to demolition, a detailed condition assessment of the bridge was undertaken (Amleh 2000; Lounis and Mirza 2001). A summary of the main data relevant to this paper is presented in the following section.

The concrete cover depth of the top mat of reinforcing steel varied considerably throughout the deck from 10 mm to 89 mm. The cover depth was normally distributed with a mean value of 36.6 mm, standard deviation of 16.5 mm, and a coefficient of variation of 45%. About 30% of the cover readings were less than the specified design cover. This high variability indicates an inadequate quality control even after considering the normal tolerances for placing reinforcement. The near-surface chloride concentration (within the first 10 mm) presented a skewed distribution approximated by a lognormal distribution with a mean of 1.27%, a standard deviation of 0.51% by weight of cement, and a coefficient of variation of 40%. The distribution of the apparent diffusion coefficient was fitted by a lognormal distribution with a mean value of 1.62×10^{-8} cm^2/s, standard deviation of 0.5×10^{-8} cm^2/s and a coefficient of variation of 30%.

Prediction of Chloride Concentration at Reinforcement Level

The results of the Monte Carlo simulation are shown in Fig.2, which illustrates the skewed distribution of chloride concentration at the reinforcement level. It follows a gamma distribution with parameters 2 and 0.783, mean value of 2.57 kg/m^3 (0.71% by cement weight), standard deviation of 1.36 kg/m^3 (0.38% by cement weight) and a coefficient of variation of 0.53. The simulation results are very close to the field measurements (Amleh 2000; Lounis and Mirza 2001), which yielded a mean value of 0.73% by cement weight and a coefficient of variation of 0.72.

Prediction of Corrosion Initiation Time

Assuming that the threshold chloride concentration (C_{th}) of the top reinforcement mat has a lognormal distribution with a mean of 0.72 kg/m^3 and coefficient of variation of 10%, the probability density of the initiation time is generated using Monte Carlo simulation as shown in Fig. 3. This skewed distribution can be approximated by a lognormal model with a mean of 10.23 years and a coefficient of variation of 1.05.

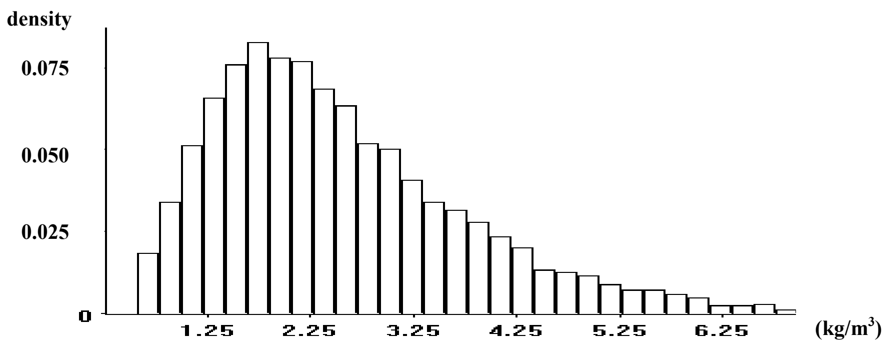

Figure 2. Distribution of chloride concentration at top steel level after 39 years

This example illustrates well the considerable scatter of the service life of concrete structures that cannot be captured by a deterministic model. Using Fig.3, the probability that the initiation time is less than or equal to 39 years is equal to 97%. However, if C_{th} is assumed lognormal with a mean of 1.44 kg/m^3 and a coefficient of variation of 10%, this probability reduces to 80%. From the different field observations and measurements made on the bridge (Amleh 2000; Lounis and Mirza 2001), including delamination, half-cell potential, chloride content, and resistivity, this probability is about 85%, which suggests that a more reliable estimate of the mean threshold content for this deck is about 1.35 kg/m^3.

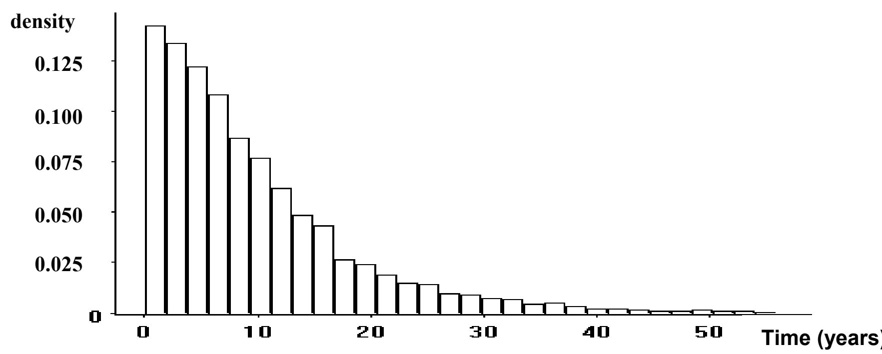

Figure 3. Distribution of corrosion initiation time of top steel

If the time to corrosion initiation (T_i) can be approximated by a lognormal distribution with mean μ_{Ti} and coefficient of variation V_{Ti}, it is possible to derive the following analytical relationship for predicting of the probability of corrosion $P_f(t)$:

$$P_f(t) = \frac{[1 - \mathrm{erf}(\frac{\beta_t}{\sqrt{2}})]}{2} \tag{5a}$$

where
$$\beta_t = \beta(t) = \frac{\ln[(\mu_{Ti} / t)\sqrt{1 + V_{Ti}^2}\,]}{\sqrt{\ln(1 + V_{Ti}^2}} \tag{5b}$$

Conclusions

This paper presented a reliability-based approach to modeling the service life of concrete structures taking into account the uncertainties in the physical modeling, and variability of the material and structural parameters affecting the corrosion process, in addition to the statistical and decision uncertainties. The application of such an approach is required in the assessment of safety and serviceability of deteriorating concrete structures in order to ensure that the probability of failure is kept at an acceptable level. A reliability-based prediction of the service life of deteriorating concrete structures provides a rational decision support tool at both the initial design stage and during the operation and maintenance stage. The implementation of such an approach allows adequate control of the safety and serviceability of the structure throughout its service life and yields low life cycle cost.

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