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Durability Monitoring for Improved Service Life Predictions of Concrete Bridge Decks in Corrosive Environments

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Abstract: The development of an effective strategy for the inspection and monitoring of the nation's critical bridges has become necessary due to aging, increased traffic loads, changing environmental conditions, and advanced deterioration. This paper presents the development of a probabilistic mechanistic modeling approach supported by durability monitoring to obtain improved predictions of service life of concrete bridge decks exposed to chlorides. The application and benefits of this approach are illustrated on a case study of a reinforced concrete barrier wall of a highway bridge monitored over ten years. It is demonstrated that service life predictions using probabilistic models calibrated with selected monitored field data can provide more reliable assessments of the probabilities of reinforcement corrosion and corrosion-induced damage compared to using deterministic models based on standard data from the literature. Such calibrated probabilistic models can help decision makers optimize intervention strategies as to how and when to repair or rehabilitate a given structure, thus improving its life cycle performance, extending its service life and reducing its life-cycle cost.

1 INTRODUCTION

A large number of reinforced concrete (RC) bridges in Canada and the northern states of USA are short and medium span bridges that exhibit serious corrosion-induced deterioration due to the use of deicing salts on roads during winter. Many of these bridges were built during the post-war

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construction booms of the 1950's, 1960's and 1970's. Approximately 25% of them are considered deficient in terms of structural capacity and functionality (U.S. DOT *et al.* 2007) as a result of increased traffic loads, changing environmental conditions, deterioration, and more stringent bridge design codes. The widespread deterioration and some recent collapses of highway bridges (Inaudi *et al.* 2009) have highlighted the importance of developing effective bridge inspection and maintenance strategies, including structural health and durability monitoring, which can help identify structural and durability problems before they become critical, endanger public safety, and impede traffic flow. The implementation of structural health monitoring programs can provide useful information on the physical health of bridges and their structural performance (Cruz and Salgado 2009, Moaveni et al. 2009, Soyoz and Feng 2009, Huang et al. 2010). Durability monitoring can supply valuable data that can be used to calibrate service life prediction models.

Currently, the majority of highway bridges are inspected at regular intervals (e.g. every 2 years) through visual inspections of the deck, superstructure and substructure, which is followed by a mapping of the observed damage to a qualitative condition rating scale (e.g. 1 to 5; 0 to 9; etc). More detailed and in-depth inspections using non-destructive evaluation methods are conducted less frequently to supplement the data obtained from visual inspection, especially for critical bridge elements to assess the level of corrosion, crack width, fatigue, spalling, delamination, stiffness, strength, etc.

1.1 Towards multi-objective management of highway bridges

Cost optimization and life-cycle cost optimization of concrete structures have been a subject of increasing research interest in recent years (Sarma and Adeli 1998, Sirca and Adeli 2005). The approaches to maintenance optimization implemented in current bridge management systems are based on single objective optimization, and more specifically on the minimization of maintenance costs or life cycle costs, which represents the present value of all the costs incurred throughout the life cycle of a bridge structure, including the costs of design, construction, maintenance, repair, rehabilitation, replacement, demolition and, in some instances, the users costs.

The management of bridges can be formulated as a multi-objective optimization problem as the bridge owner or manager seeks to satisfy implicitly and simultaneously several objectives, such as the minimization of costs to owners and users, improvement of public safety and security, improvement of serviceability and functionality, minimization of maintenance time, minimization of traffic disruption, etc. The solution of such a bridge management problem can be obtained by using the techniques of multi-criteria or multi-objective optimization. Despite its relevance, the use of risk as a criterion for decision-making in the management of highway bridges is not easy given the complexities of assessing the probability of failure, as well as the consequences of failure (Wenzel 2009). In life cycle cost and cost-benefit analyses, which have been implemented in many bridge management systems currently used by different transportation agencies in North America, all losses, including fatalities, injuries, and social costs are quantified in monetary terms, including the cost of human life.

It is clear from the above discussion that the existing approaches to decision making have serious limitations as they express all losses in monetary terms and consider only one criterion at a time, e.g. minimization of owner costs. Considering this issue, Lounis *et al.* (1995, 2000, 2009) proposed a multi-objective optimization approach for decision-making, which can incorporate all relevant objectives, in order to help solve the bridge management problem. Such an approach enables a better evaluation of the effectiveness of preservation and protection strategies in terms of several objectives (safety, security, mobility, economy, cost, etc.) and determines the optimal solution that achieves the best trade-off between all of them (including conflicting ones, such as safety and cost). Figure 1 outlines a framework for the multi-objective management of highway bridge structures. The development and implementation of this framework will provide effective decision support to bridge owners and managers in optimizing the allocation of limited funds, as well as improving the life cycle performance of their bridges.

Structural health monitoring (SHM) is one of the major components of this framework, which appears in two distinct phases: (i) Risk-based assessment of the current condition of the structure, in combination with non destructive evaluation and load rating, which will help assess the physical conditions of the bridge; and (ii) Risk management of identified safety-critical and security-critical bridges, along with other parallel activities such as visual inspection, non-destructive evaluation, quantitative assessment, maintenance, strengthening, and protection. The complete framework and multi-objective approach for the effective management of bridges, which are outside the scope of this paper, are presented in Lounis *et al.* (2009).

1.2 Towards structural health and durability monitoring of highway bridges

Structural health and durability monitoring, either with embedded sensors or by actual field testing, is an evolving technology that can be used to monitor the condition of existing or new civil engineering structures. Depending on the importance and location of the structure, available budgets and monitoring objectives, three major types of SHM can be adopted: (i) occasional short-term monitoring, to confirm design assumptions, monitor the structural behaviour during a planned exceptional event, or help in the planning of a more complex form of monitoring; (ii) periodic monitoring, to assess the parameters that are mainly influenced by cyclic events; and (iii) continuous monitoring, to assess the performance over the life cycle of a critical structure, including parameters that are influenced by static and dynamic events (Braunstein *et al.* 2002). Continuous remote monitoring may be more cost-effective in the long term than conducting frequent field testing, considering the cost of labour, the costs to the users, and their safety. In-depth information on the design of SHM systems and applications on bridge can be found elsewhere (Mufti 2001, Braunstein *et al.* 2002, Glisic & Inaudi 2007, Frangopol *et al.* 2008, Cardini & Dewold 2009).

The key benefits of SHM, including the monitoring of structural performance or long-term durability, can be summarized as follows:

(i) *Improvement of public safety and extension of service life of bridges* – SHM can enable bridge owners to remotely monitor the performance of their structures from a central site via Internet, thereby reducing the number of site visits for visual inspections and destructive- and/or non-destructive testing. Therefore, SHM can provide continuous information on structural performance (e.g. excessive deformations and stresses, yielding, foundation settlement) and durability (e.g. concrete cracking, reinforcement corrosion, freeze-thaw damage). The access to such continuous information, rather than relying on periodic information (e.g. every 2nd year from conventional visual inspections) will allow timely decisions on corrective measures before problems become critical and endanger public safety. Safety-critical bridges should be monitored to ensure that the likelihood of failure of critical load-bearing elements is kept very low, especially for bridges in major urban centres, and non-redundant bridge systems, for which failure can have catastrophic consequences.

(ii) *Development and calibration of service life prediction models* – The vital information acquired by SHM can foster a better understanding of damage initiation and damage accumulation in bridge structures (Cusson *et al.* 2006; Butcher & Newhook 2009) and calibration of service life prediction models. In addition to the scarcity of field data available from full-size engineering structures, many key parameters used in service life prediction models are highly variable and uncertain, which may adversely affect the reliability of the predictions. The continuous updating of service life prediction models with selected field data can have a considerable impact on the planning of preservation actions.

(iii) *Improved design and rehabilitation of concrete structures* – Another benefit of SHM is to help re-assess the current live loads and environmental loads on bridge structures. This will help identify any major variations from the historical values that are assumed in bridge design codes, thus reducing the uncertainty of critical design parameters. This is becoming an important issue given the growing concerns with climate change and its potential impact on the safety and serviceability of bridge structures due to increases in wind loads, flooding, thermal gradients, freeze-thaw cycles, deicing salt use, etc. Structural health monitoring can also be used to ensure the structural integrity of the bridge during the sequential rehabilitation or replacement of damaged load-bearing elements (Carrion *et al.* 2009).

(iv) *Continuous security monitoring of critical bridges* – A continuous video camera surveillance system can be installed to monitor critical bridges and their immediate surroundings in order to identify and respond to potential threats to the bridges and their components. Various types of activities can be monitored depending on the type of bridge, such as car and truck traffic, movement of people, as well as boats, ships and aircraft activities. Computer vision and pattern recognition technology can also be used to allow computers to process recorded images, watch for danger signs, and send alarms to security officers. Such technologies require high-speed communication systems to deliver the information to remote security offices for analysis, response, reporting and archiving purposes. It is possible though to process and archive images on site; however, this would not allow post-catastrophe analysis (FHWA 2009).

(v) *Faster development and acceptance of new construction technologies* – SHM can provide a structured approach to assess the performance of emerging innovative technologies applied in demonstration projects (thus promoting innovation), whether they are conducted on old structures or new construction. Research on bridges involves the use of sensors and data

1.3 Objectives

The objectives of this paper are twofold: (i) to present a probabilistic mechanistic modeling approach based on durability monitoring to assess the life cycle performance of concrete bridge decks in corrosive environments; and (ii) to demonstrate the effective use of selected data obtained from field monitoring to update and improve the accuracy of mechanistic service life prediction models. A case study of the monitoring of a concrete highway bridge barrier wall is presented and used to illustrate the approach and its benefits.

2 MECHANISTIC MODELING OF SERVICE LIFE OF RC BRIDGE DECKS SUBJECT TO CHLORIDE ATTACK

2.1 Proposed durability monitoring and mechanistic modeling approach

Corrosion of reinforcing and prestressing steel is a major problem for bridges in Canada and the northern states of the USA. The deterioration induced by corrosion accounts for a large portion of the maintenance expenditures of several bridge owners (U.S. DOT *et al.* 2007). For the structural performance and safety assessment of bridges, several models have been developed in the past, including Markovian models (which are only qualitative), deterministic mechanistic models, and probabilistic mechanistic models. However, for the durability assessment and service life prediction of bridge decks in corrosive environments, fewer models have been proposed and a large number of them are deterministic models that do not consider the uncertainty and variability associated with the key parameters governing the initiation of corrosion and corrosion-induced damage processes.

The proposed approach consists of the monitoring of critical durability parameters that govern the proposed mechanistic models for the service life of RC bridge decks exposed to chlorides. The field monitoring could be periodic or continuous, using sensors and/or non destructive evaluation (NDE) methods ideally, or even limited destructive testing (e.g. cores), if absolutely necessary. The monitoring program could be conducted over a few years or during the whole service life, depending on the importance of the structure, the available monitoring budget and other factors. Although long-term continuous monitoring is preferable, it will be shown in a case study (Section 3) that periodic monitoring of a new structure conducted over the first ten years of its life can provide reliable input data for the prediction of its remaining service life (prediction models to be presented later in this section).

For the case of RC bridge decks exposed to chlorides from deicing salts or seawater, four variables related to concrete and steel properties have strong influences on service life, namely:

(i) surface chloride content of concrete, which may increase over time and vary in space due to the continual use of deicing salts on roads, precipitation, and drainage;

(ii) chloride diffusion coefficient of concrete, which may decrease over time due to the continuing cement hydration, decreasing concrete porosity, which may also vary in space;

(iii) chloride threshold of the reinforcement, which may vary in time and space. It is also highly uncertain as it depends on environmental conditions and properties of concrete and steel;

(iv) corrosion rate of the reinforcement and rate of deterioration of the concrete structure, which are both highly variable.

The considerable uncertainties associated with these parameters governing the service life of RC bridges highlight the need for calibrating and updating service life prediction models with selected field monitoring data, and for the use of probabilistic analysis methods (to be presented next).

In addition to the high variability and uncertainty of these parameters, this problem is compounded by the fact that just a few of these key parameters can actually be monitored by the use of remote sensors. Although some analog sensors may be used to determine the chloride threshold (indirectly) and the corrosion rate of the reinforcement, no commercial sensors are currently available to remotely determine the chloride content and chloride diffusion in concrete. Therefore, there is a strong need for the development and acceptance of new sensors for durability monitoring in order to reduce the frequency of required field testing, including the use of destructive testing (such as taking concrete cores for the determination of chloride content profiles in a structure).

2.2 Conceptual two-stage service life model

Figure 2 presents the different corrosion-induced damage mechanisms developing in a typical reinforced concrete bridge deck exposed to chlorides, which identifies two distinct stages: a corrosion initiation stage, and a propagation stage. This is a modified version of Tuutti's simplified model (1982), which did not consider the effect of early-age cracking on chloride penetration during the corrosion initiation stage. With time, each stage develops into higher levels of damage, which include: (i) early-age cracking of concrete due to restrained shrinkage (if any); (ii) initiation of reinforcement corrosion after a relatively long period of chloride diffusion through concrete; (iii) internal cracking around the reinforcing bars due to the build-up of corrosion products; (iv) surface cracking due to further progression of corrosion-induced cracks; (v) spalling or delamination of the concrete cover; and finally (vi) rehabilitation or replacement of the concrete deck, depending on the amount of concrete damage to the deck that can be tolerated by the bridge owner. Note that other conceptual service life models do exist, such as that proposed by Li *et al.* (2007), based on crack width and sectional strength, as opposed to Tuutti's model which is based on the degree of corrosion and damage accumulation.

Based on the conceptual model illustrated in Figure 2, the service life (t_s) can be defined as the total time to reach a given corrosion-induced damage level, which is the sum of the corrosion initiation time (t_i) (also considering the effect of early age cracking), and the propagation time (t_p) corresponding to a given level of damage, as follows:

$$t_s = t_i + t_p \tag{1}$$

Several different estimates of service life can be obtained by using this conceptual model. In North America for instance, different departments of transportation have adopted different criteria for defining the end of service life as a measure to limit excessive damage to the structure and as an indicator for the need of a major rehabilitation. For example, the end of service life of a concrete highway bridge deck is generally defined as the time to reach critical damage levels in terms of spalling or delamination (e.g. AASHTO 2002, Table 1). It is up to the bridge authority to decide which criteria to use based on many factors, including the desired levels of serviceability, functionality and safety, the costs of maintenance and repair, and available funds. In this paper, the end of service life of a concrete bridge deck is defined as the time to onset of spalling. It is not the intent of this paper to relate the end of service life of a bridge to the time of possible collapse, since the focus of the proposed approach is on durability of a bridge deck under service conditions, and not the structural integrity of the bridge.

2.3 Prediction of chloride ingress into concrete

In uncracked concrete, the chloride ingress can be determined by using Crank's solution of Fick's second law of diffusion (Crank 1975):

$$C(x,t) = C_s\left(1 - erf\left(0.5\, x / \sqrt{D_c\, t}\right)\right) \tag{2}$$

where $C(x,t)$ is the chloride content at depth x after time t; C_s is the apparent surface chloride content (assumed constant); D_c is the apparent chloride diffusion coefficient (assumed constant); and *erf* is the error function. The above model also assumes that concrete is homogenous and diffusion is the main mechanism of chloride ingress into concrete. In porous solids such as concrete, chlorides can penetrate into concrete via different physical mechanisms, such as: diffusion, capillary absorption, electrical migration, and permeation due to hydraulic pressure heads, depending on the exposure condition and moisture content (Kropp & Hilsdorf 1995). The lack of accuracy of Fick's model has been recognized for a long time, since it has been used with the so-called "apparent" values of the surface chloride content and chloride diffusion coefficient, which are obtained by fitting the Fickian model to available field data. However, this model has gained wide acceptance due to its simplicity and practicality (Bamforth 1998; Tang & Gulikers 2007).

In cracked concrete, chloride ingress through cracks may lead to premature reinforcement corrosion and concrete deterioration (De Schutter 1999; Rodriguez & Hooton 2003). In order to model the effect of cracking on chloride ingress, it is generally accepted that the chloride diffusion coefficient corresponding to uncracked concrete need to be modified into an apparent value representing the cracked medium (François & Arliguie 1999; Gérard & Marchand 2000). Boulfiza et al. (2003) used a simplified smeared approach to estimate the effect of cracks on chloride ingress. It assumes that chloride ingress into cracked concrete can be approximated by using Fick's second law of diffusion, in which an apparent diffusion coefficient (D_{app}) is found with this equation:

$$D_{app} = D_c + \frac{w_{cr}}{s_{cr}} D_{cr} \tag{3}$$

where D_c is the chloride diffusion coefficient in uncracked concrete; w_{cr} is the crack width; s_{cr} is the crack spacing; and D_{cr} is the diffusion coefficient inside the crack, which may be assumed to be $5\text{x}10^{-10}$ m^2/s, according to Boulfiza et al. (2003).

2.4 Prediction of onset of corrosion and concrete spalling

In order to predict the time of corrosion initiation (t_i), Equation 2 can be rearranged by setting $C(x,t)$ equal to the chloride threshold (C_{th}), at which steel corrosion is expected to initiate, and x equal to the effective cover depth of the reinforcement (d_c), as follows:

$$t_i = \frac{d_c^2}{4D_{app}\left[erf^{-1}\left(1-\frac{C_{th}}{C_s}\right)\right]^2} \tag{4}$$

The accumulation of corrosion products over time generates contact pressure between the rebar and the surrounding concrete, and may initiate cracks if the tensile stress in the concrete reaches the tensile strength (f'_t). The concrete cover and the corroding rebar can be modelled as a thick-wall cylinder subjected to a uniformly distributed internal pressure (p_i), assuming that concrete is a homogeneous elastic material, and that corrosion products are equally distributed around the perimeter of the reinforcing bars (Bažant 1979, Tepfers 1979). The radial stress (σ_r) and tangential stress (σ_t) generated in the concrete cover by the expansion of the corrosion products can be estimated using the thick-wall cylinder model (Figure 3, Timoshenko 1956). This model allows the calculation of the increases in the rebar diameter (Δd) related to different stages of corrosion-induced damage (i.e. internal cracking, surface cracking, spalling, or delamination) by calculating the radial displacements at the inner surface of the cylinder. Assuming that the external pressure induced by external loads on the structure is small or negligible, the increase in rebar diameter at the onset of spalling is given by:

$$\Delta d = \frac{p_i}{E_c}\left[1+\nu+\frac{d^2}{2d_c(d_c+d)}\right]d \tag{5}$$

where d is the rebar diameter, d_c is the concrete cover thickness, ν is Poisson ratio of concrete, E_c is the effective elastic modulus of concrete taking into account the effect of creep (Bažant 1979). In the case of spalling, assuming a 45° plane of failure with respect to the concrete surface, the internal pressure in the concrete before reaching the concrete tensile strength (f'_t) is given by (Bažant 1979):

$$p_i = 2\, f_t^{'} \left(\frac{d_c}{d} + 0.5 \right) \tag{6}$$

Note that similar expressions of Δd and p_i have been developed to model the other limit states illustrated in Figure 2 (i.e. onset of internal cracking, onset of surface cracking, and onset of delamination), and can be found elsewhere (Lounis & Daigle 2008).

Assuming that, for a row of parallel reinforcing bars, the corrosion products are smeared over the entire area, thus the production rate of corrosion products per unit area is given by:

$$j_r = \frac{m_r}{s\, t_p} \tag{7}$$

where s is the rebar spacing, t_p is the propagation time, and m_r is the mass of corrosion products defined by Bažant (1979) as:

$$m_r = \frac{\pi\, d\, (\Delta d)}{2\left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \tag{8}$$

where ρ_r is the density of corrosion products (assumed at 3600 kg/m^3 for $Fe(OH)_3$); ρ_s is the density of steel (7860 kg/m^3); and α is the molecular weight ratio of metal iron to the corrosion product (assumed at 0.52).

Substituting Equation 8 into Equation 7 and solving for t_p, the propagation time corresponding to the onset of spalling can be determined as follows:

$$t_p = \frac{\pi\, d\, (\Delta d)}{2\, s\, j_r \left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \tag{9}$$

The production rate of corrosion products (j_r) is also related to the corrosion current density (i_{cor}), which can be monitored in the field, and the loss of rebar diameter due to corrosion, which was determined as $0.023\, i_{cor}\, t_p$ by Andrade & Alonso (2001). Consequently, j_r can also be expressed as:

$$j_r = \frac{0.023\,\pi\, d\, I_{cor}}{2s\left[\dfrac{1}{\rho_r} - \dfrac{\alpha}{\rho_s}\right]} \tag{10}$$

where the parameter 0.023 is an empirical conversion factor from current density units of $\mu A/cm^2$ to corrosion rate units of mm/year.

2.5 Modeling of uncertainty in service life prediction

Service life predictions may be obtained through a simple deterministic analysis, i.e. an analysis that does not consider the variability and uncertainty associated with the main parameters governing the different mechanisms of deterioration in RC bridge decks. Deterministic analyses are based on mean or characteristic values of the variables and can only predict the times to reach the different stages of corrosion initiation and corrosion-induced damage caused by an average condition. In order to account for the variability and uncertainty of the input parameters, more reliable service life predictions can be obtained by using reliability-based methods, with the key input parameters expressed in terms of their mean values and coefficients of variation (COV).

If the investigated governing parameters exhibit considerable spatial variability within the given bridge deck, the analysis may require the use of random fields instead of random variables for their modeling (Vanmarcke 1983; Stewart *et al.* 2003; Lounis & Daigle 2008). The use of random fields, however, requires considerably more input data such as the correlation function and scale of fluctuation (which are most often not available in practice), and adds considerable analytical and computational complexity to the analysis.

If the variability of the parameters can be assumed to be completely random, i.e. no systematic variability and no spatial correlation, then the use of random variables for modeling the above parameters is adequate (Lounis & Daigle 2008). Considering that the RC bridge deck can be divided into a very large number of small surface areas with same probability ($P\%$) of spalling after a number of years, it can be approximated that a certain percentage ($P\%$) of them are spalled after a given time. These probabilities can be used to make initial and approximate comparisons to the five condition states defined in the 2002 AASHTO guidelines for the assessment of concrete bridge decks (Table 1), which is used in the bridge management systems of several states in the USA and some provinces in Canada.

In general, the time-dependent probability of failure (i.e. reaching limit states such as critical chloride content, onset of corrosion, onset of cracking, spalling or delamination) can be formulated as follows:

$$P_f(t) = P(t_s < t) \tag{11}$$

The determination of the time-dependent probability of failure is a very complex problem due to the high level of nonlinearity of the performance functions that describe the above limit states, which can be modelled with 1st order reliability methods, 2nd order reliability methods, Monte Carlo simulation, or random fields. In this approach, the advanced first-order reliability method was selected and used to determine the probability of failure for the above described service life performance functions. The determination of the probability of failure can be formulated as a nonlinear optimization problem, in which the solution minimizing the distance from the origin to the failure surface (described by any of the above performance functions) is sought. This minimum distance was defined as a measure of the reliability index by Hasofer & Lind (1974). Once the reliability index (β) is determined, the probability of failure can be estimated as follows:

$$P_f(t) = 1 - \Phi(\beta) \tag{12}$$

where Φ is the well-known standard normal cumulative distribution function. An iterative numerical algorithm and a software called SLAB-D were developed to solve the above nonlinear optimization problem and to determine the reliability index (Lounis & Daigle 2008). If the random variables are not normal, then the above approach provides estimates of the probability of failure (Madsen *et al.* 1986). The validity of this approach will be demonstrated later in the case study by comparing its prediction to that obtained with a Monte Carlo simulation (Melchers 1999).

2.6 Implementation of proposed approach in practice

The use of detailed reliability-based mechanistic models for every asset of a large network of bridges can become very costly and unmanageable. To address the need of asset managers for decision support tools that are both practical and reliable, Lounis and Madanat (2002) proposed a two-level decision process based on two types of deterioration models. At level-1, the simplified Markovian-type cumulative damage models are used to forecast the overall deterioration and required maintenance funds for both short and long term planning for a network of bridges, and to identify the critically damaged systems. At level-2, reliability-based mechanistic deterioration models (such as the ones proposed in this paper) are used for the analysis of critical assets, which were identified from the first level of management analysis or from specific detailed assessment. In the case of an RC bridge deck exposed to chlorides, the analysis at level-2 can provide quantitative estimates of the damage level and relevant information to select the most cost-effective maintenance strategies for the deteriorated system.

In the next section, the proposed approach will be demonstrated in a case study of a reconstructed concrete bridge barrier wall of a highway bridge located near Montreal, Canada. It should be emphasized however that the approach can be applied on other members of a bridge, regardless of surface orientation or the type and severity of the exposure conditions. As long as the chloride content profile of the concrete and corrosion rate of the reinforcement can be measured or monitored either periodically or continuously, then accurate predictions can be achieved with the proposed prediction approach.

Ideally embedded sensors, or other non destructive evaluation methods (such as proposed by Sengul and Gjorv 2008), should be used to obtain the desired information to update the prediction models presented above. In cases where sensors or NDE could not be used for any reason, the bridge authority will have to weigh out the impact of using minimal destructive testing at carefully selected locations on the bridge deck (e.g. coring through membrane in deck slab) against the risk of relying on less accurate service life predictions. Alternatively, another option may be considered by the bridge authority when dealing with limited or partial data. For example, Choi and Seo (2009) proposed an assessment procedure for corrosion-damaged structures based on the imprecise reliability theory, which was developed for cases where only insufficient data are available.

3 CASE STUDY: LIFE CYCLE PERFORMANCE PREDICTION OF A CONCRETE BRIDGE BARRIER WALL USING DURABILITY MONITORING

The purpose of this section is to illustrate the benefits of durability monitoring for improving service life predictions, and to point out that some of the input data that are commonly assumed in typical prediction models can be somewhat different from reality and lead to inaccurate predictions, as some parameters are uncertain and can also vary widely in space and time.

3.1 Description of bridge deck and experimental program

In 1996, the Ministry of Transportation of Québec undertook the rehabilitation of the Vachon bridge, which is a major highway bridge in Laval (near Montréal) Canada. Part of the rehabilitation consisted of rebuilding the severely corroded concrete barrier walls, of which ten 34-m spans were selected for the application and evaluation of different corrosion inhibiting systems. The barrier wall reinforcement consisted of 15-mm diameter bars, with eight longitudinal bars distributed in the cross-section, and transverse reinforcement spaced at 230 mm, as illustrated in Figure 4. The concrete had a water-cement ratio (w/c) of 0.36 to ensure low permeability, a cement content of 450 kg/m^3, and a mean 28-day strength of 45 MPa. On-site surveys of the barrier wall were performed annually from 1997 to 2006 (in May/June to ensure similar environmental conditions), including measurements of corrosion potential and corrosion rate of the reinforcement, and concrete electrical resistivity in the barrier wall, of which the concrete cover was 75 mm. For early detection of corrosion, sets of rebar ladders were embedded during construction. The ladder bars had concrete cover thicknesses of 13 mm, 25 mm, 38 mm, and 50 mm (Figure 4), allowing additional corrosion measurements to be taken. More details can be found in Cusson & Qian (2009).

3.2 Measurement and prediction of chloride ingress into concrete

Concrete cores were taken from the bridge barrier walls after 1, 2, 4, 5, 8 and 10 years of exposure to deicing salts in order to test several parameters, including chloride content. The total acid-soluble chloride contents at different depths in the concrete were measured according to the ASTM C114 (1999) standard procedure. Of the 10 spans of barrier wall under study, three of them had identical concrete mix designs and similar concrete surface conditions (referred to as Spans 12, 19 and 21 according to the bridge deck construction drawings). Figure 5 presents the profile of the total chloride content measured in the concrete after 10 years of exposure to deicing salts (each data point is a mean value obtained from two cores). The best-fit curve was obtained by a regression analysis using Crank's solution of Fick's 2nd law of diffusion (Equation 2). For the regression, values of C_s and D_c were adjusted until the sum of errors between the solution and the field data reached a minimum. The chloride contents measured at a mean depth of 6 mm from the concrete surface were not considered in the regression analysis because diffusion near the surface is not the only main mechanism governing chloride ingress in concrete (as discussed above).

From the field data obtained over a period of 10 years, a mean apparent surface chloride content of 22.7 kg/m^3 and a mean apparent chloride diffusion coefficient of 0.63 cm^2/year were determined. In reality, the highest near-surface chloride content was measured to be at least 16.5 kg/m^3 in the barrier wall (Figure 5), which is already much higher than the maximum value of 8.9 kg/m^3 suggested by Weyers (1998) for geographical regions where severe exposures to deicing salts are expected (Table 2). It is noted that these guidelines were developed for bridge decks located in the USA and may not apply to regions like Canada or other northern countries, where more deicing salts may be used during longer and colder winter periods.

Similarly, the mean value of the apparent chloride diffusion coefficient of 0.63 cm^2/year determined after 10 years for the concrete barrier wall was found to be larger than those obtained from the literature for a concrete with similar *w/c*. For example, Figure 5 shows the predicted chloride profiles using Equation 2 with C_s = 8.9 kg/m^3, as suggested by Weyers (1998) for severe exposure conditions, and (a): D_c = 0.21 cm^2/year based on data provided by Dhir *et al.* (1990) on a concrete very similar to that used in this case study, or (b): D_c = 0.38 cm^2/year determined with the empirical model of Boulfiza *et al.* (2003) for a concrete with a *w/c* = 0.36 containing no supplementary cementing materials.

It can be seen that the chloride profiles are largely underestimated when compared to the measured field data, even for a short period of 10 years. These discrepancies can be explained by the large fluctuations of many factors influencing chloride ingress into concrete, including concrete mixture formulation, hydration and curing characteristics, temperature and humidity conditions, and surface chloride content. It may be concluded that the determination of the chloride profile for a given concrete structure, using selected literature values from similar concretes tested under similar environments, may result in inaccurate estimates, leading to potential overestimations of the remaining service life of the structure.

Figure 6 presents the evolution of the apparent surface chloride content over time, where it is shown to increase significantly over time up to a maximum value of 22.7 kg/m^3 after 10 years. Figure 6 also presents the time variation of the apparent chloride diffusion coefficient, where it is observed to decrease by a factor of 2 from Year 2 to Year 10. This reduction could be explained in part by the continuing cement hydration and corresponding reduction in concrete porosity.

Considering that most chloride diffusion prediction models use constant values of C_s and D_c from the literature or initial field data, the above observations suggest that prediction models may also yield inaccurate predictions if input values of C_s and D_c are not regularly updated with field monitoring data. In order to investigate this point further, the time variation of the chloride content was estimated for three cases: Case 1 using constant values of C_s and D_c from the literature for a similar concrete; Case 2 using the varying values of C_s and D_c determined from field data measured over 10 years; and Case 3 using the constant values of C_s and D_c determined from field data measured at a time of 10 years. Table 3 gives the values used in this comparative analysis.

Since varying values of C_s and D_c were used in Case 2, Equation 2 could not be used to determine the chloride profile over time. Instead, it was determined by solving the one-dimensional partial differential equation of Fick's 2nd law of diffusion:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left(D_c \frac{\partial C}{\partial x}\right) \tag{13}$$

The numerical solution of Equation 13 was obtained with a one-dimensional model of the concrete barrier wall using the finite difference method with the following central operator:

$$C_{i,\Delta t} = C_i + 0.25\left[C_{i-1} - 2C_i + C_{i+1}\right] \qquad \text{and} \qquad \Delta t = \frac{\Delta x^2}{4D_c} \tag{14}$$

where $C_{i,\Delta t}$ is the chloride content in concrete at the i^{th} layer for a given time increment Δt; and Δx is the thickness of the concrete layer (which was set to 5 mm in this case). Note that this particular diffusion problem could also be solved in a similar manner using other techniques, for example, cellular automata, which is a special class of evolutionary algorithms (Vořechovská *et al.* 2009).

Figure 7 presents the estimated chloride contents at different concrete depths over time for Case 1, in which the values of C_s and D_c were selected from the literature for a similar concrete in a similar environment. For example, depending on the selected value of D_c, a chloride content of 1.8 kg/m^3 (C_{th} according to CEB 1992) would take 20 to 35 years to accumulate in this concrete at a 50 mm depth, or 45 to 80 years at a 75 mm depth. Figure 8 presents the same calculations for Case 2, in which varying and more severe values of C_s and D_c based on field measurements were used. It can be seen that the predicted chloride contents are much higher than those from Case 1. For example, in Case 2, a chloride content of 1.8 kg/m^3 would take only 5 years to accumulate in this concrete at a 50 mm depth, or only 12 years at a 75 mm depth, which is quite different from the predictions for Case 1.

In order to demonstrate the effect of using constant vs. varying input values on the predictions of chloride content, the results of Case 3 (using the constant field values of C_s and D_c determined at an age of 10 years) are compared in Figure 8 to those of Case 2 (using the varying field values determined over a period of 10 years). It can be seen that chloride profiles are slightly different in the very early years; however, not enough to invalidate the predictions of the models using constant values of C_s and D_c. In this case study, when using low values of chloride threshold (e.g. 1.8 kg/m^3), Case 2 predictions are slightly more conservative than Case 3 predictions by only 2 or 3 years. Predictions of high chloride contents obtained in the long term do not seem to be significantly affected by the choice of input values from Case 2 or Case 3, as long as the values are based on field measurements made on the structure of interest.

The above analysis clearly illustrates the need for field monitoring to update current prediction models for which the accuracy of the predictions rely on the quality of the input data. For example, Figure 9 compares the measured chloride profile obtained at an age of 10 years (data points) to three estimated chloride profiles for an age of 10 years (curves) using Equation 2 updated with C_s and D_c values measured at ages of 2 years, 5 years and 10 years. It can be observed that the use of more recent updates of C_s and D_c (e.g. Year 5 values instead of Year 2 values) in the chloride diffusion model can considerably improve the prediction of the long-term chloride profile (in this case 10 years). It is also interesting to note that predictions using the Year 2 data in this case study provided a much better estimate of the long-term chloride profile than the predictions based on the literature values selected for a similar concrete under similar exposure conditions (see Figure 5).

3.3 Observation and prediction of corrosion initiation and concrete spalling

Figure 10 presents a sensitivity analysis of the times to initiate corrosion and concrete spalling (using Equations 1 to 10), depending on several factors: (i) cover thickness; (ii) chloride threshold; and (iii) corrosion rate. At the 75 mm depth (location of the main reinforcing bars in the barrier wall), the prediction indicates a time to corrosion initiation of 15 years based on the threshold value suggested by CEB (1992). No significant corrosion was in fact observed on sections of reinforcing bars cut from the barrier wall after 10 years (Cusson & Qian 2009).

The literature shows a strong disagreement amongst researchers on the range of values to use for the chloride threshold of conventional reinforcing steel in concrete. For example, the following values have been reported: 0.6-4.9 kg/m^3 (Stratfull *et al.* 1975), 0.7-5.25 kg/m^3 (Vassie 1984), 6.3-7.7 kg/m^3 (Lukas 1985), 0.87 kg/m^3 (West & Hime 1985), and 1.05-1.75 kg/m^3 (Henriksen 1993). There are many reasons explaining these differences, including the variability of several concrete properties and exposure conditions, such as pH level, w/c, type and content of cementing materials, temperature, relative humidity (Tuutti 1993), as well as the ambiguity associated with the definition of chloride threshold (Alonso *et al.* 2000). Another factor affecting the prediction accuracy is the high variability of the corrosion rate, which depends on many factors, including the location on the reinforcement relative to a crack, and the yearly variations of ambient temperature, oxygen content and ambient relative humidity (Shiessl & Raupach 1990). Over a year, the corrosion rate may be expected to reach maximum values during the warm season and minimum values during the cold season, and possibly negligible values if the concrete is frozen.

At a depth of 75 mm in the barrier wall, the models predicted concrete spalling to occur after 20 years of exposure (Figure 10), based on the moderate corrosion rate (I_{cor}) of 0.50 $\mu A/cm^2$ (Cox *et al.* 1997) and on the CEB (1992) chloride threshold value. In fact, the mean corrosion rates measured after 10 years in the bridge barrier walls (near cracks) were 0.25 $\mu A/cm^2$ for the 75-mm deep main reinforcement and 0.30 $\mu A/cm^2$ for the 25-mm deep test rebars. With this field data, and assuming a minimum chloride threshold of 1.8 kg/m^3, the models predicted the onset of spalling after at least 25 years for the 75-mm deep bars.

As mentioned above, the considerable uncertainty and variability associated with the key parameters governing the service life of RC bridges under corrosive environments require the use of probabilistic analysis methods for a reliable prediction of their remaining service life. The prediction models presented above in Section 2 require input data for different parameters related to the properties of concrete, steel, the environment and geometry of the reinforcement. From the data acquired during the field study at the Vachon bridge, it was possible to estimate the average values and coefficients of variation of many key parameters of interest. This information is presented in Table 4. It can be seen that different parameters have quite different coefficients of variations. For instance, bar spacing had the smallest COV of 4% because it is related to geometry, thus to the quality of workmanship. The corrosion rate had the largest COV of 40% as it varies widely in space and time due to a number of different factors as mentioned earlier. Figure 11 presents histograms of the distribution of rebar spacing and corrosion rate measured in the barrier wall. From this information, the assumption of normal distribution in a probabilistic analysis is found reasonable.

Finally, Figure 12 presents the probabilities of steel corrosion and concrete spalling for depths of 25 mm and 75 mm in the bridge barrier wall. For these calculations, the models (Equations 1 to 12) were used and fed with input values determined from the field data obtained after 10 year of exposure to deicing salts. The mean value of the chloride threshold to initiate reinforcement corrosion was assumed to be 1.8 kg/m^3 (converted from the CEB 1992 threshold value of 0.4% by mass of cement). Also, one of the curves determined by the models presented in this paper was recalculated using the well-known Monte-Carlo simulation technique using 50,000 iterations. The purpose of this comparison was to verify the validity of the advanced first-order reliability method (Section 2.5) for solving nonlinear problems.

As expected, the results show that as time increases, the probabilities of initiating corrosion and concrete spalling increase, and more rapidly for the thinner concrete cover and for the higher assumed corrosion rate of the steel reinforcement. As mentioned before, these probabilities may be used by bridge inspectors to assess the condition of the deck by adapting to existing guidelines (e.g. AASHTO 2002) and to decide upon the best time to take corrective actions (local repair, major rehabilitation, or replacement). For example, if the bridge owner's policy is to never let a bridge deck (or any part of it) deteriorate more than Condition State 3 (Table 1), the maximum probability of spalling could be set to 25% before undertaking any major corrective actions. In this case, a bridge deck, having concrete and environment that are similar to those of the investigated concrete barrier wall, would reach Condition State 3 after 18 years if the actual corrosion rate was 0.25 μA/cm^2, or 14 years if the corrosion rate was 0.50 μA/cm^2. An optimal decision could then be made by selecting the preservation action that yields the lowest life cycle cost (Lounis & Daigle 2008).

4 SUMMARY AND CONCLUSIONS

The corrosion-induced deterioration of highway bridges can have serious consequences in terms of reduced safety, serviceability and durability. This paper proposed a probabilistic modeling approach based on durability monitoring for improving the life cycle performance predictions of aging concrete bridge decks built in corrosive environments. Its application and benefits were demonstrated on a case study of rebuilt RC barrier walls on a highway bridge near Montreal, Canada. The following conclusions and recommendations are suggested:

- The mechanistic models of the proposed approach require input data on four main variables (i.e. surface chloride content, chloride diffusion coefficient, chloride threshold, and corrosion rate) for the determination of different limit states along the life cycle of a bridge deck, including early-age cracking, onset of corrosion, internal/surface cracking, spalling and delamination.
- The above variables and other key parameters (e.g. concrete cover thickness) do vary considerably in space and/or time, and some of these cannot be measured accurately. Relatively high coefficients of variation between 20% and 40% have been measured in the field.
- Given the considerable uncertainties associated with these parameters governing the service life of RC bridges exposed to chlorides, the use of probabilistic analysis methods are required.
- The advanced first order reliability method is a simple and practical approach that enables the modeling of uncertainties in service life prediction and requires a minimum amount of data. The validity of this simplified approach was demonstrated by a comparison with the Monte Carlo simulation method.
- In the case study, it was shown that some of the key input data that are commonly used in service life prediction models could be very different from reality due to their high variability and uncertainty. For example, values of apparent chloride content in concrete were measured up to 23 kg/m^3, which is about 2.5 times higher than the maximum values suggested in the literature for the most severe exposure to deicing salts.

- It was also shown that values of apparent surface chloride content in concrete increases over time while values of apparent chloride diffusion coefficient decreases over time, contrasting with some common assumptions used in service life modeling. However, it was shown that the use of constant values measured at the age of 10 years could provide comparable estimates of service life compared to models using varying values.
- It was demonstrated that service life predictions could be improved significantly by updating the models with selected field monitoring data from embedded sensors or on-site corrosion surveys, as opposed to using selected data from the literature. The proposed approach can be used on any RC elements of bridge decks as long as the governing corrosion parameters could be monitored on site and fed to the probabilistic mechanistic prediction models.
- Recommendations were provided for the application of the proposed approach to a given network of bridges. A two-level decision process based on two types of deterioration models was suggested, in which critically damaged bridges are first identified by using simplified Markovian cumulative damage models, and then individually analysed using the proposed durability monitoring and probabilistic mechanistic modeling approach.

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List of Tables

Figure Captions

Table 1. AASHTO description of condition states for concrete bridge decks with no surface protection and built with uncoated reinforcement.

Condition state	**Description**
1	The surface of the deck has no patched area and no spall in the deck surface.
2	The combined distress area (i.e. existing patches, spalling and delamination) of the deck is less than 10%.
3	The combined distress area of the deck is between 10% and 25%.
4	The combined distress area of the deck is between 25% and 50%.
5	The combined distress area of the deck is more than 50%.

Table 2. Severity classification of corrosive environment (Weyers 1998).

Severity	**Apparent surface chloride content**
Low	0.0-2.4 kg/m^3
Medium	2.4-4.7 kg/m^3
High	4.7-5.9 kg/m^3
Severe	5.9-8.9 kg/m^3

Table 3. Mean values of apparent surface chloride contents and apparent chloride diffusion coefficients used in comparative analysis.

<table>
<tr><th rowspan="2">Parameter</th><th>Selected data from literature</th><th colspan="2">Selected data from field measurements (see fig. 6)</th></tr>
<tr><th>(Case 1)</th><th>(Case 2)</th><th>(Case 3)</th></tr>
<tr><td>C_s</td><td>8.9 kg/m^3</td><td>- increasing from 10.8 to 22.7 kg/m^3 in 10 yrs, and
- constant at 22.7 kg/m^3 for the remaining time</td><td>22.7 kg/m^3</td></tr>
<tr><td>D_c</td><td>0.21 cm^2/yr, or 0.38 cm^2/yr</td><td>- decreasing from 1.17 to 0.63 cm^2/yr in 10 yrs, and
- constant at 0.63 cm^2/yr for the remaining time</td><td>0.63 cm^2/yr</td></tr>
</table>

Table 4. Mean values and coefficients of variation used in the probabilistic analysis.

Parameter	Mean	COV (%)	Description of data used for calculation of Mean and COV values
C_s (kg/m^3)	See Table 3 (C. 2), or Figure 6	14	16 measurements taken in barrier wall after 10 years (excluding test sections with surface coating/sealer)
D_c (cm^2/yr)	See Table 3 (C. 2), or Figure 6	28	16 measurements taken in barrier wall after 10 years (excluding test sections with surface coating/sealer)
C_{th} (kg/m^3)	1.8	30	No field data; Mean value suggested by CEB 1992; Assuming COV for high variability
d_c (mm)	75	20	4 measurements along height of barrier wall
S (mm)	230	4	57 longitudinal measurements between stirrups of barrier wall (see Figure 11a)
I_{cor} (μA/cm^2)	0.25	40	52 measurements taken in barrier wall at cracks after 10 years (see Figure 11b)

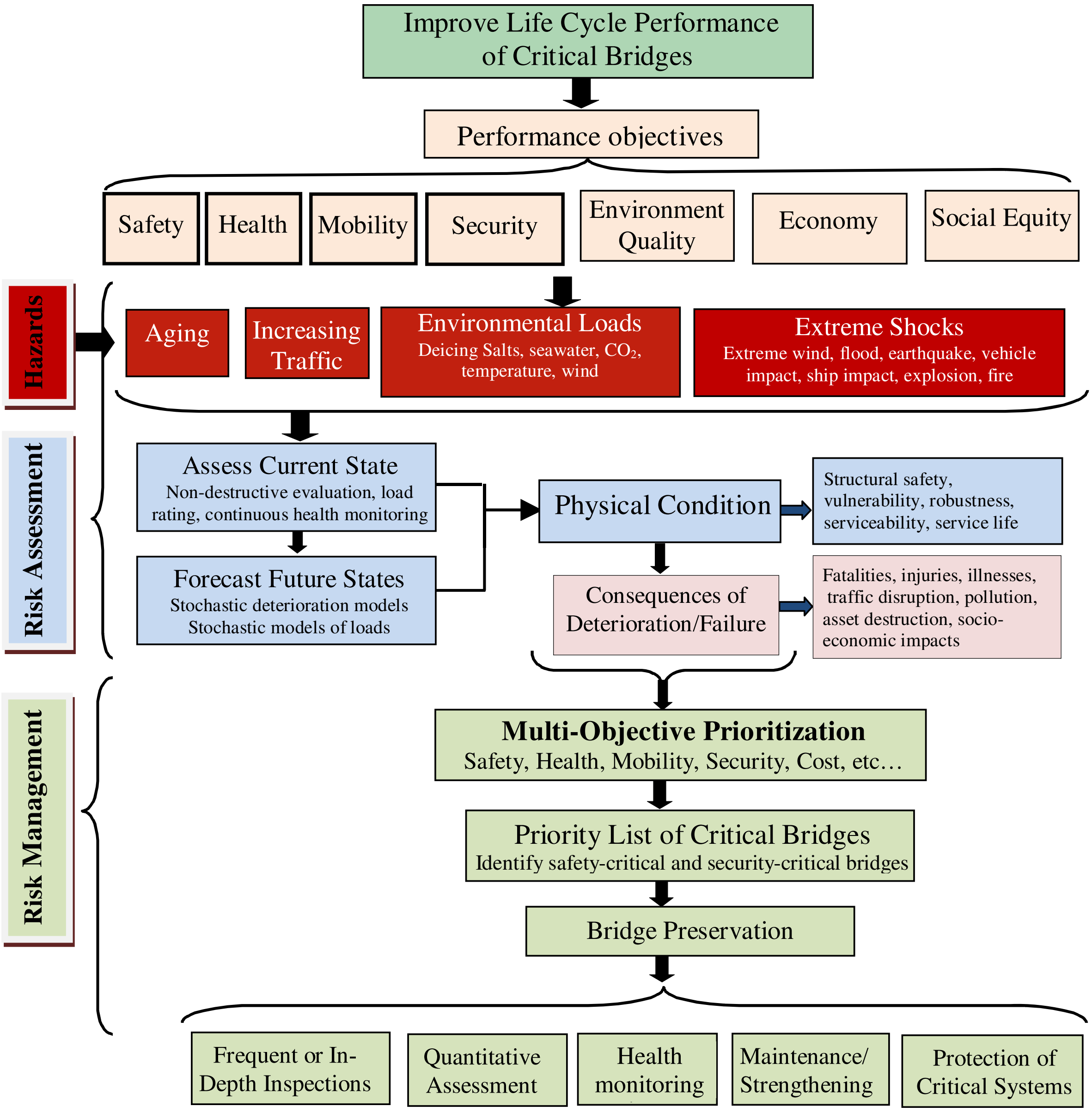

Fig. 1. Framework for multi-objective management of aging highway bridges

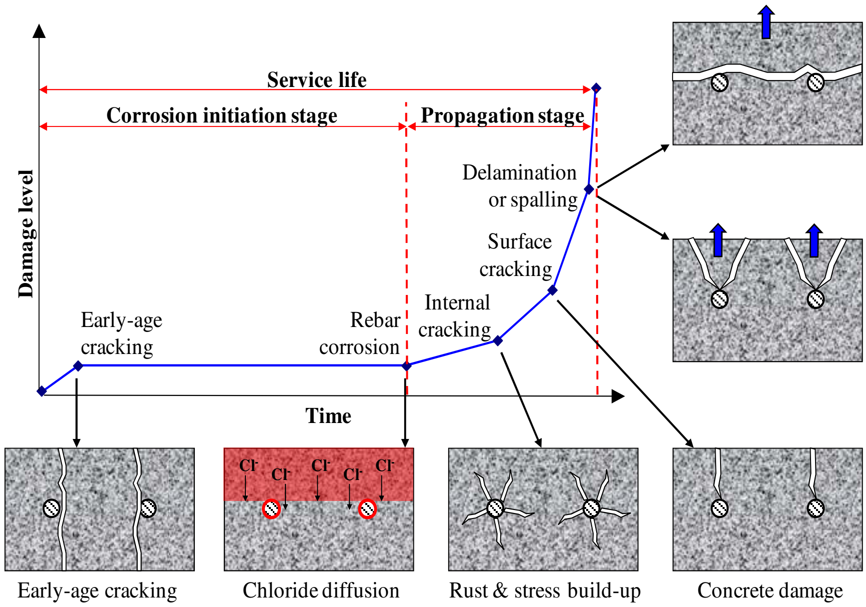

Fig. 2. Schematic description of service life model.

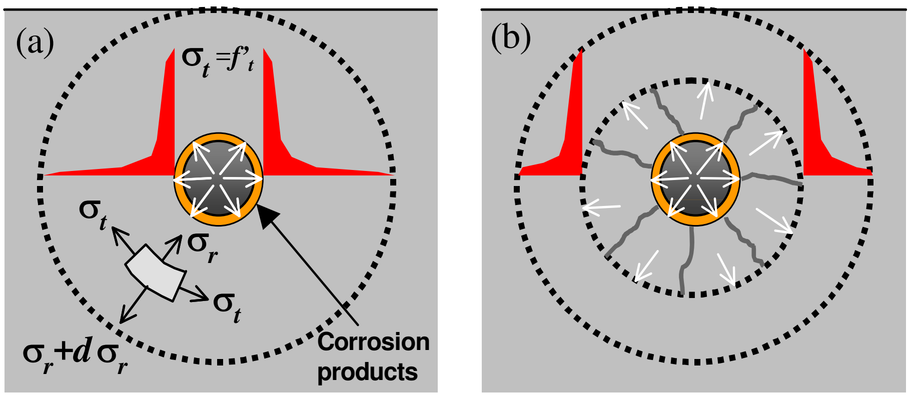

Fig. 3. Uncracked and cracked thick-wall cylinder models of corroding RC bridges:
(a) Tensile stresses developed at crack initiation;
(b) Propagation of internal cracks in thick-wall cylinder;
(adapted from Lounis & Daigle 2008).

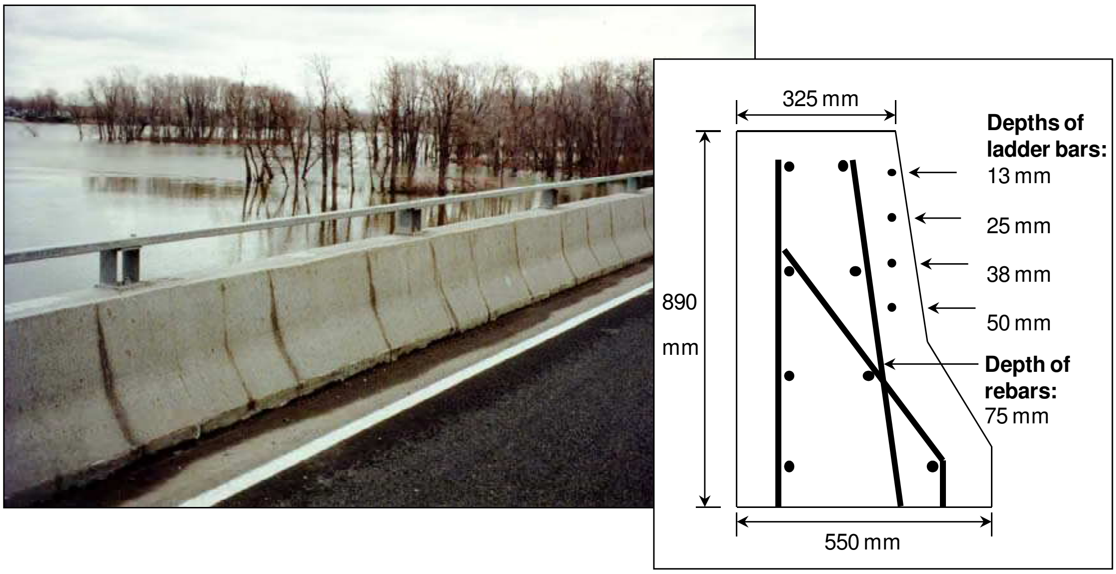

Fig. 4. Cross-section of reconstructed RC bridge barrier wall (Vachon bridge, Laval, Canada).

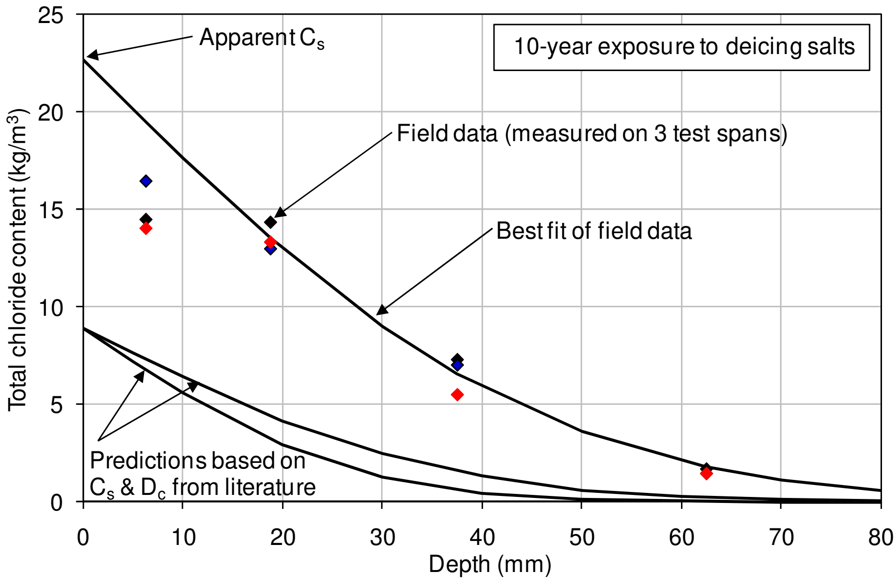

Fig. 5. Measured and predicted profiles of total chloride content after 10 years.

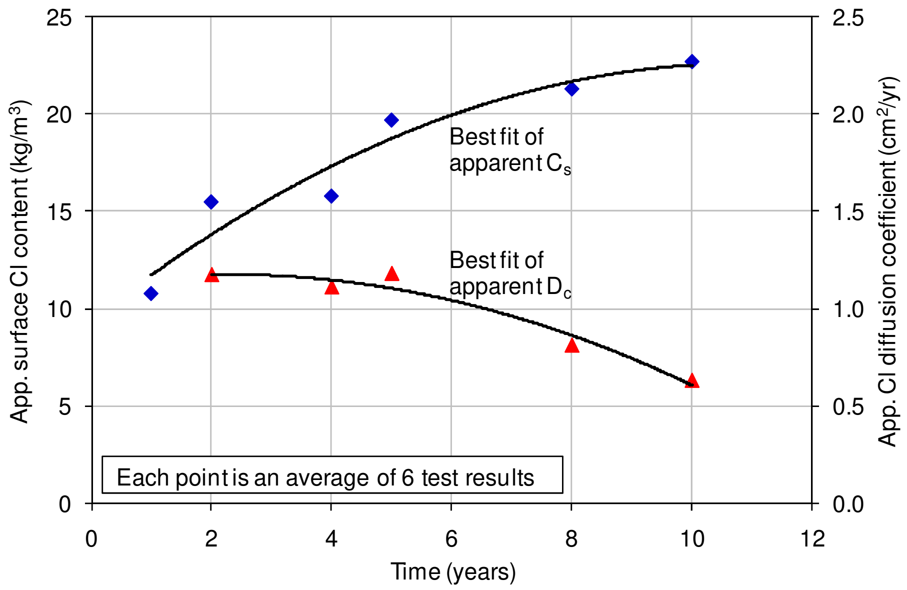

Fig. 6. Apparent surface chloride contents and apparent chloride diffusion coefficients determined over a period of 10 years.

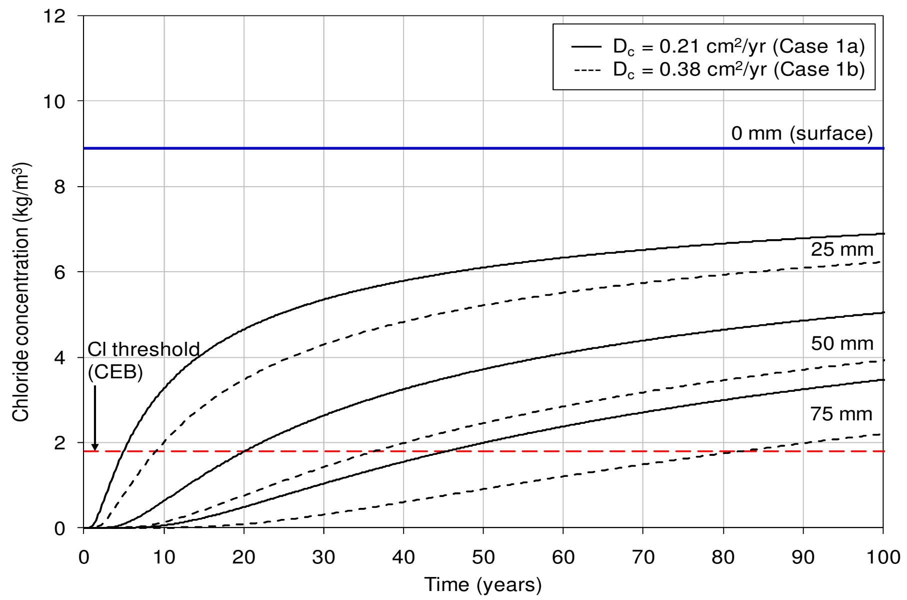

Fig. 7. Predictions of chloride profiles for Cases 1a and 1b (with C_s and D_c selected from literature).

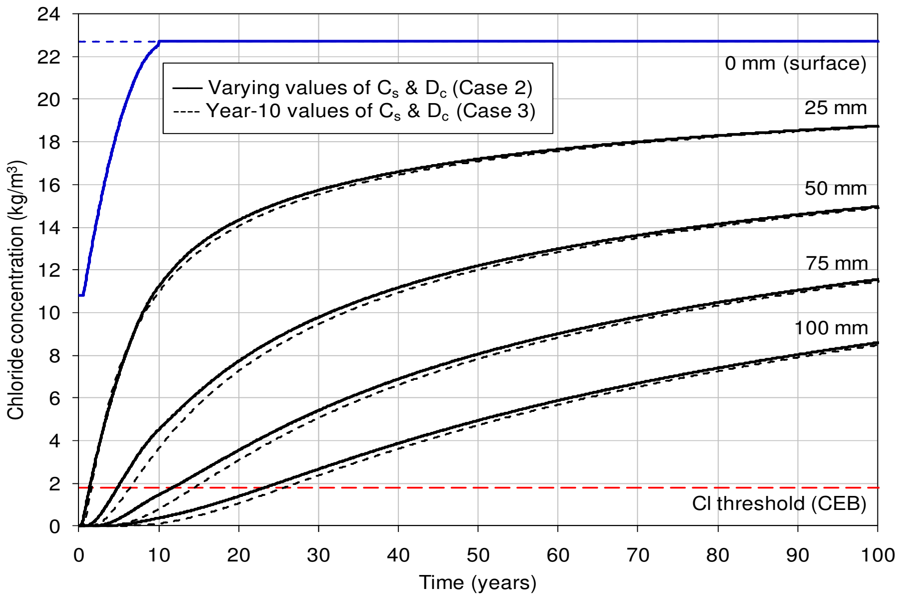

Fig. 8. Predictions of chloride profiles for Cases 2 & 3
(with C_s and D_c obtained from durability monitoring of bridge).

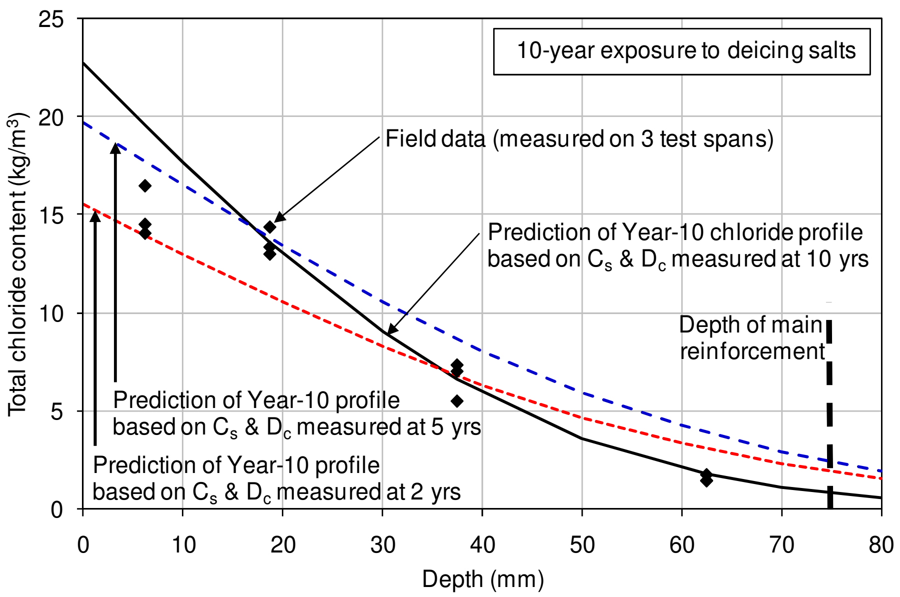

Fig. 9. Predictions of 10-year chloride profiles based on different measurement times.

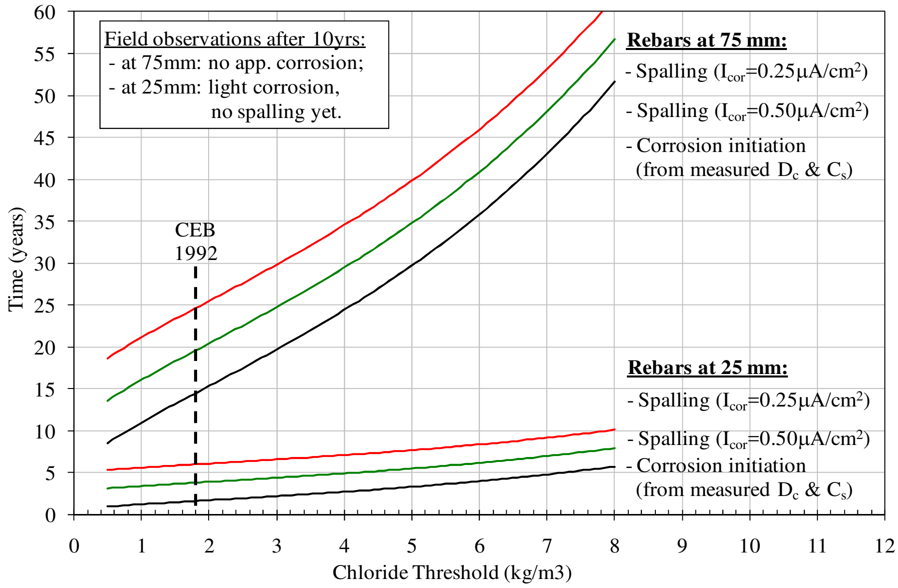

Fig. 10. Sensitivity analysis of time to rebar corrosion and time to concrete spalling.

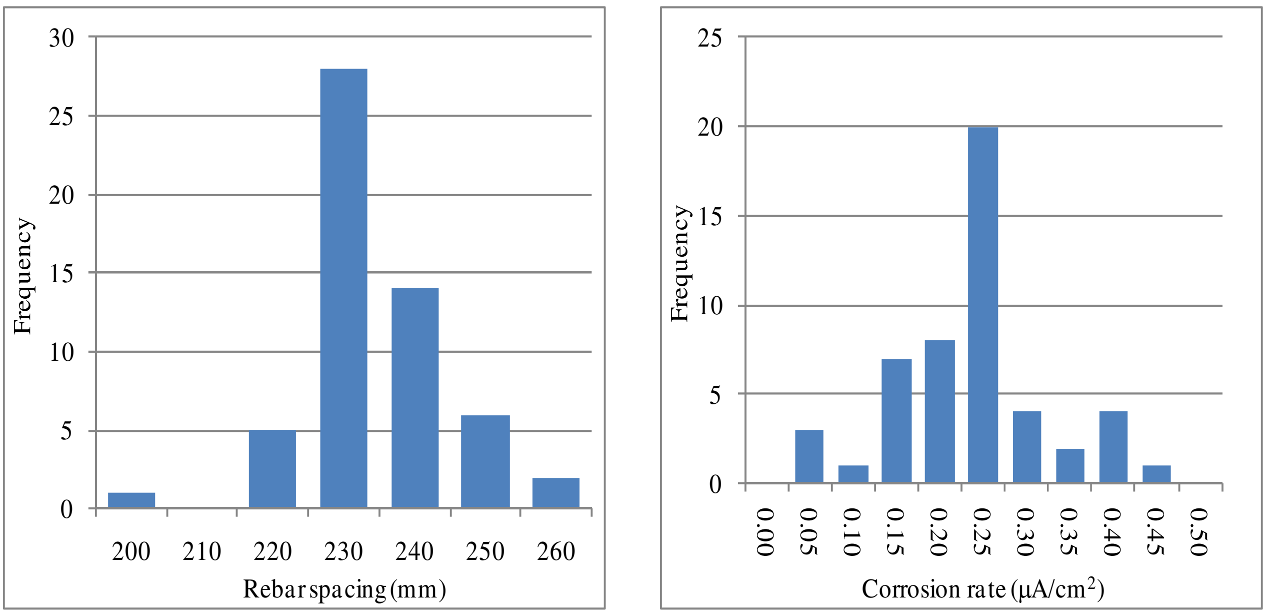

Fig. 11. Histograms of two selected parameters measured in barrier wall:
(a) rebar spacing between stirrups; (b) current density at transverse shrinkage cracks after 10 years.

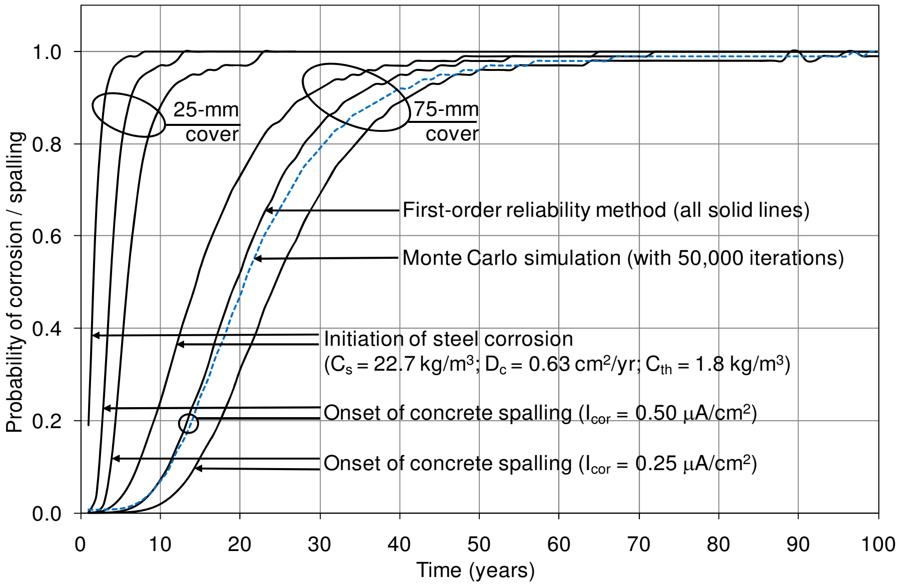

Fig. 12. Estimated probability of steel corrosion and concrete spalling for concrete cover depths of 25 mm and 75 mm.