Structural Safety and Serviceability of Concrete Bridges Subject to Corrosion

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Abstract: A structural reliability analysis model is developed to include interaction between transverse cracking, diffusion of chlorides, and corrosion initiation; influence of design specifications on corrosion initiation and propagation; and serviceability limit states (e.g., spalling). The reliability model is used to evaluate probabilities of structural and serviceability failures for flexure and spalling limit states, for a typical reinforced concrete bridge continuous slab. Chloride contamination will occur from the application of deicing salts. It was confirmed that the application of deicing salts caused a significant reduction in structural and serviceability reliabilities; this observation is in agreement with field data of bridge performance. Moreover, the reliability analysis allowed the effect of corrosion to be measured in a quantitative manner. The influence of concrete cover and specified concrete compressive strength was found to be particularly significant on the probability of spalling. The reliability analysis is used to demonstrate how known exceedence of a serviceability limit state (spalling) can be used to update the probability of structural failure. The time-dependent reliability analysis developed in the present paper may, at a later stage, be applied to other reinforced concrete bridge structural configurations or be used as the reliability module in existing inspection, maintenance, or load rating procedures, i.e., bridge management systems.

Introduction

Corrosion of reinforcement bars in reinforced concrete (RC) structures is the primary cause of structural deterioration of bridge decks. Corrosion-initiated longitudinal cracking and associated spalling of the concrete cover are also common problems. Corrosion is initiated mainly by chloride contamination, often in conjunction with inadequate cover or poor quality concrete. The deterioration process associated with the corrosion of reinforcement has the following two stages:

1. Initiation—time to commencement of reinforcement corrosion. Corrosion is initiated mainly by chloride contamination, often in conjunction with reduced concrete cover, low quality concretes that are unsuitable for their exposure environment, and poor compaction and curing. Chlorides may diffuse through the protective concrete cover, and corrosion is initiated once the chloride concentration exceeds a critical threshold value. Corrosion also may be initiated if drying shrinkage, flexural, thermal, or other crack widths are sufficiently large to allow the direct ingress of chlorides, oxygen, and moisture. Carbonation (exposure to atmospheric CO_2) will free chlorides bound to the concrete matrix, thus making the reinforcement more susceptible to chloride-induced corrosion.
2. Propagation—reinforcement corrodes causing loss of area (metal loss). The increased volume of corrosion products (rust) causes concrete tensile stresses that may be sufficiently large to cause internal microcracking, external longitudinal cracking, and eventually spalling. This may lead to an acceleration in corrosion rate and/or reduction of bond, which in turn may lead to serviceability failure and/or a loss of structural integrity.

Bridges constitute a considerable investment in infrastructure and directly affect the productivity of the transportation industry. However, a large proportion of bridges are old and in need of repair; for instance, in the United States more than 50% of bridges are over 50 years old. The rate of structural deterioration increases with bridge age; this is caused by an increase in legal load standards, deterioration of bridge materials, wear and damage from other traffic, poor or inadequate maintenance, and other factors. It is also due to the increase in concrete chloride concentrations due to increased use of deicing salts—<1,000,000 t/year in the early 1950s to ~15,000,000 t/year in the 1990s in the United States [e.g., Baboian (1995)]. It is therefore not surprising that in the United States, approximately 150–200 bridges/year suffer either partial or full collapse and that 125,000 bridges in the United States are rated as structurally deficient (bridges that are restricted to light vehicles only are closed or require immediate rehabilitation to remain open). It has been estimated that $90 billion is needed to rectify these problems. This is in addition to the $140 billion currently being spent to maintain this infrastructure at its existing level (Dunker and Rabbat 1993; Aktan et al. 1996). Similar problems exist in many other countries. There is obviously a strong financial incentive that the existing bridge infrastructure be conserved and that new bridges will require minimum maintenance or repairs over their service life.

Bridge management systems are used for the optimal allocation of resources for bridge design, construction, and maintenance. If all parameters influencing bridge performance are known (or deterministic) then the decision-making process associated with the optimal allocation of resources should be relatively straightforward. In practice, however, uncertainties arise in predicting service or lifetime loads; when using nonexact structural behavior (prediction) models; in considering the variability in material properties, workmanship, element dimensions, environmental conditions, inspection data, and maintenance; in predicting fatigue, corrosion, or other deterioration processes; etc. Consequently, decisions related to bridge design and assessment are based on uncertain or incomplete information. In such cases, a useful decision-making tool is reliability (or probabilistic) analysis since this can provide a rational criterion for the comparison of the likely consequences of decisions taken under uncertainty. Bridge reliabilities may be used to

- Evaluate performance by comparing with reliability-based acceptance criteria such as a target reliability index

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J. Infrastruct. Syst. 1998.4:146-155.

- Estimate cost-effectiveness of decisions using a risk-cost-benefit analysis
- Prioritize bridge maintenance or repair activities by ranking reliabilities of different bridges
- Identify the most likely failure mode(s) within a structure or structural system

For example, an updated reliability of a bridge (after an inspection) may be compared with some minimum acceptable reliability; from this comparison the relative safety of the bridge can be ascertained, a load rating assigned, or a "residual life" estimated [e.g., Stewart (1998)]. The present paper develops a preliminary probabilistic framework for one such time-dependent life-cycle (reliability) analysis.

Research into probabilistic methods for the evaluation of deterioration of RC bridges is proceeding at an increasing rate [e.g., Das (1996) and Frangopol and Hearn (1996)], although it is still relatively limited. Hoffman and Weyers (1996) developed a probabilistic model for bridge deck chloride diffusion due to the application of deicing salts. However, this model assumes that failure occurs when corrosion is initiated —a very conservative assumption. Thoft-Christensen et al. (1996) developed what appears to be the first reliability-based expert system that calculates structural reliabilities considering both collapse and flexural cracking limit states based on initiation (chlorides, carbonation) and propagation of corrosion of the reinforcement and decision models for inspection and repair. This has significant potential; however, the time-invariant reliability analysis used is nonconservative, and the effects of shrinkage and longitudinal (spalling) cracking, serviceability limit states, and loss of bond are ignored. Val et al. (1998) considered reduction in area of steel and loss of bond using a nonlinear finite-element model, deflection limit states, immediate corrosion initiation, and both homogeneous (carbonation) and localized (chlorides) corrosion propagation models. Frangopol et al. (1997) developed a useful practical application of probabilistic corrosion modeling; namely, a reliability-based design approach based on the minimization of expected lifetime costs. Stewart and Rosowsky (1998b) developed a structural deterioration reliability model to calculate probabilities of structural failure (flexure) for a reinforced concrete continuous slab bridge. In that study, corrosion was initiated from the application of deicing salts, atmospheric exposure in a marine environment (sea spray), carbonation, and flexural cracking.

Existing bridge reliability studies have tended to ignore the interaction between transverse cracking, diffusion of chlorides, and corrosion initiation; influence of design specifications on corrosion initiation and propagation; and serviceability limit states (e.g., spalling). Particular emphasis is placed on modeling these phenomena in the present study. The structural deterioration reliability model developed by Stewart and Rosowsky (1998b) is extended to include these phenomenon. Described herein is an improved probabilistic framework to calculate probabilities of structural and serviceability failures (flexure and spalling limit states) for a typical RC bridge continuous slab. Chloride contamination will occur from the application of deicing salts. The model accounts for the variability in chloride diffusion, critical threshold chloride concentration, corrosion rates, concrete material properties, element dimensions, reinforcement placement, environmental conditions, and loads. It is assumed that corrosion will lead to a reduction in the cross-sectional area of the reinforcing steel that may then lead to longitudinal cracking or spalling of the concrete cover. Transverse cracking (shrinkage, flexure) is also considered as a cause of initiation. The influence of design variables such as the water-to-cement ratio, concrete compressive strength, and cover are included in the material behavior models. Loss of bond is not considered as it appears not to have a significant effect on the reliability of bridge elements in flexure (Val et al. 1998). Monte Carlo simulation is used to calculate time-dependent probabilities of structural failure and spalling for annual increments over the lifetime of the bridge (100 years). The reliability analysis is also used (1) to demonstrate how known exceedance of a serviceability limit state (spalling) can be used to update the probability of structural failure; and (2) to estimate the probability that the bridge will fail in subsequent years given that it has survived T years of service loads [i.e., reliability increases for service proven (older) structures]. The influence of concrete cover and specified concrete compressive strength on reliabilities is also assessed. The reliability analysis is used to demonstrate how known exceedance of a serviceability limit state (spalling) can be used to update the probability of structural failure. The time-dependent reliability analysis developed in the present paper may, at a later stage, be applied to other RC bridge structural configurations or be used as the reliability module in existing inspection, maintenance, or load rating procedures (i.e., bridge management systems).

INITIATION OF CORROSION

Chloride Penetration due to Application of Deicing Salts

Numerous studies have found that the penetration of chlorides through concrete is best represented by a diffusion process if the concrete is assumed to be relatively moist. In this case, the penetration of chlorides is given empirically by Fick's second law of diffusion, expressed as

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \tag{1}$$

where C = chloride ion concentration at a distance x from the surface at t years; and D = apparent diffusion coefficient. In a physical sense, however, field conditions deviate significantly from the assumptions implicit in Fick's law. For instance, the cover is not always saturated with water, and so chloride ions penetrate concrete by diffusion and advection provided by the penetrating moisture front (Vitharana 1997). Concrete is not homogeneous due to the presence of microcracking, interconnected pores, and aggregate particles; the diffusion coefficient will change with time as hydration proceeds, etc. [e.g., Detwiler et al. (1997)]. Hence, Fick's law is not a good model of this phenomenon. Nonetheless, Fick's law is often used since in many cases the diffusion equation provides the "best fit" to laboratory or field data. Clearly, predictions using this approach are valid only if best-fit parameter values are applied to structures with similar material, environmental, and field conditions.

In using Fick's law, it is preferable that concentrations be given in terms of water-soluble chlorides since it is generally accepted that corrosion is influenced by the free (water-soluble) chloride concentration present in the concrete pore solution [e.g., Tuutti (1982)]. However, nearly all chloride concentration data given in the literature refers to acid-soluble chloride concentrations; hence, chloride concentrations described herein also refer to acid-soluble (or total) chlorides.

In a study comprising samples taken from 321 concrete bridge decks in the United States, Hoffman and Weyers (1994) concluded that the surface chloride concentration does not accumulate with time. Instead, they found that the surface chloride concentration is in equilibrium with the concentration of chlorides in the saturated deicing salt solution and so is constant. In this case, the chloride content $[C(x, t)]$ at a distance x from the concrete surface at time t is given by

J. Infrastruct. Syst. 1998.4:146-155.

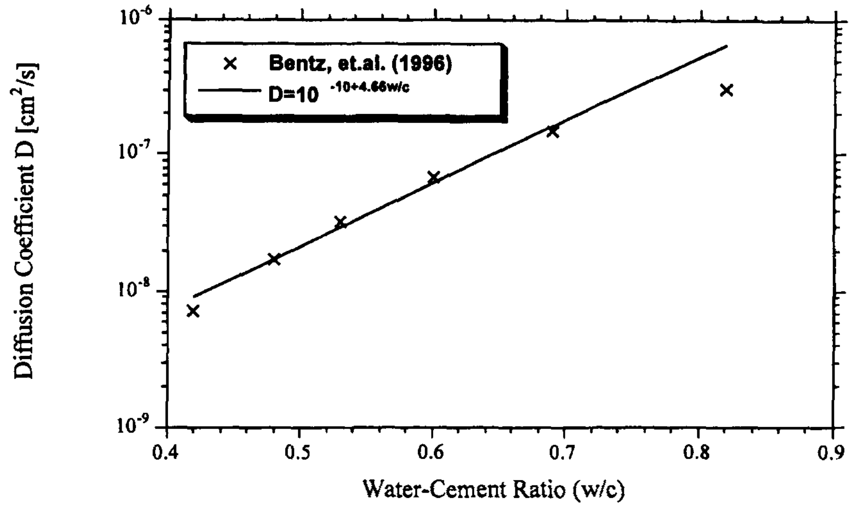

FIG. 1. Relationship between Water-To-Cement Ratio and Diffusion Coefficient

$$C(x, t) = C_0 \left[1 - \operatorname{erf}\left(\frac{x}{2\sqrt{tD}}\right)\right] \quad (2)$$

where C_0 = surface chloride content; D = apparent diffusion coefficient; and erf = error function.

Estimates of the mean diffusion coefficient for each state in the United States ranged from 0.6E-8 cm^2/s to 7.5E-8 cm^2/s, with an overall mean of approximately 2.0E-8 cm^2/s (Hoffman and Weyers 1994). This study further found that the coefficient of variation for diffusion coefficients obtained for each state varied from 0.3 to 1.6, with a mean of 0.75 representing variability within a "typical" state of the United States. This estimate is in broad agreement with Suzuki et al. (1990) who found that the coefficient of variation for diffusion coefficients for structures in coastal areas in Japan is 0.56. Note that these statistical analyses were conducted for a range of different structures and so represent the variability associated with different design parameters (e.g., mix designs, curing regimes), exposures, etc. In the present study, a coefficient of variation of 0.75 is assumed. To avoid negative values the diffusion coefficient is modeled as a lognormal random variable.

From the same field data, the mean surface chloride content for each state in the United States varied from 1.2 to 8.2 kg/m^3. The mean and coefficient of variation of surface chloride content (for the United States) is C_0 = 3.5 kg/m^3 and 0.5, respectively, and is modeled as a lognormal distribution (Hoffman and Weyers 1994).

Concrete permeability is influenced by mix proportions (water-to-cement ratio, cement type), curing, compaction, and environment. However, it is well accepted that the water-to-cement ratio (mix proportion) has a significant influence on concrete permeability since an increase in the water-to-cement ratio increases capillary porosity [e.g., *Durable* (1992)]. The influence of the water-to-cement ratio on the diffusion coefficient D was obtained from a computer-integrated knowledge system developed by Bentz et al. (1996). The system developed predictive models for chloride diffusion from 16 separate sources of experimental data. As such, for a given mix proportion, the knowledge system can predict the diffusion coefficient (Fig. 1). A least-square line of best fit of the predicted diffusion coefficients gives

$$D \approx 10^{-10+4.66w/c} \quad (3)$$

where D is in square centimeters per second. For a typical water-to-cement ratio of 0.5, the diffusion coefficient is approximately 2.0E-8 cm^2/s, and so (3) is consistent with the statistical analysis of bridge diffusion coefficients reported by Hoffman and Weyers (1994). The water-to-cement ratio is estimated from Bolomey's formula; namely

$$w/c \approx \frac{27}{f'_{cyl} + 13.5} \quad (4)$$

where f'_{cyl} = concrete compressive strength of a standard test cylinder (MPa).

The chloride concentration must reach a critical threshold chloride concentration Cr to cause dissolution of the protective passive film around the reinforcement, thereby initiating corrosion of reinforcement. Numerous studies have shown that the critical threshold chloride concentration tends to lie within the range of 0.6–1.2 kg/m^3 [e.g., Cady and Weyers (1983) and Mehta (1991)]. The critical threshold chloride concentration is influenced by, among other things, water-to-cement ratio, cement type, temperature, water and oxygen content, pH, fly ash content, and silica fume content. Thus it is to be expected that the critical threshold chloride concentration will vary, even for similar structures. In this study, the critical threshold chloride concentration is modeled as a uniformly distributed random variable within the range of 0.6–1.2 kg/m^3 (Hoffman and Weyers 1996).

A comparison of chloride contents in RC elements (at a depth of 50 mm) is shown in Fig. 2 for surfaces exposed to deicing salts and specified concrete compressive strengths of 20.7 MPa (3,000 psi), 27.6 MPa (4,000 psi), and 34.5 MPa (5,000 psi). It is observed that concrete compressive strength has a significant influence on diffusion coefficient and, hence, chloride concentration.

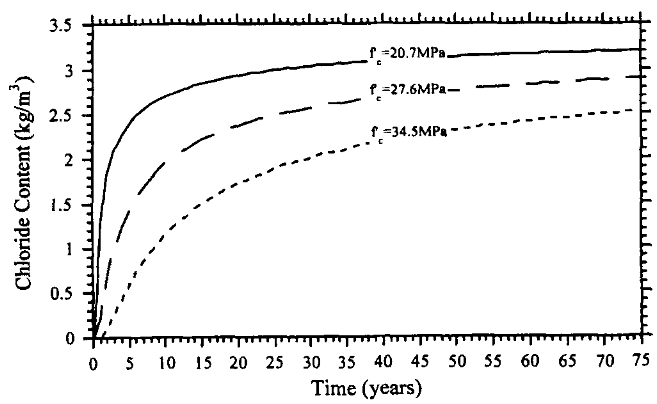

FIG. 2. Chloride Concentrations for RC Surfaces for Deicing Salts at Depth of 50 m

Transverse Cracking—Flexure and Shrinkage

Cracks exist in concrete due to construction (processing, placement, curing, early-age loading, etc.), shrinkage, and thermal effects. Crack initiation and propagation is also influenced by type of structural system; for example, cracking phenomena will differ between RC slab bridges and bridges with RC slabs supported by stringers. A common cause of cracking in hardened concrete is drying shrinkage ("Guide" 1996). In addition, load-induced tensile stress fields cause internal cracks to propagate outward and existing external cracks to propagate inward. Large surface cracks are undesirable since it is probable that wider cracks permit greater migration of moisture, oxygen, and carbon dioxide through the concrete cover and toward the reinforcing steel. These are the necessary ingredients for corrosion. It is likely that the diffusion process described previously includes (though not explicitly) the influence of microcracking.

The term transverse cracking used herein will refer to intersecting cracks, caused by drying shrinkage or flexure, crossing the main reinforcement. In general, it can be stated that crack width may affect the initiation of corrosion, but corrosion is confined to the cracked zone and does not affect the subsequent propagation process [Beeby 1983; Concrete Society (CS) 1995; Schießl and Raupach 1997]. Longitudinal (or bond) cracking will influence corrosion propagation and so will lead to spalling and strength loss (Beeby 1983; CS 1995)—longitudinal cracking and spalling will be considered later in this paper.

A number of flexural crack width models have been proposed, many of which have been used as the basis of code provisions aimed at the control of macrocracking. Unfortunately, many existing crack control equations [ACI 318 ("Building" 1995) and BS8110 ("Structural" 1985)] imply significantly increased crack widths as concrete cover increases (e.g., 80–90% increase in crack width for doubling of cover). Yet experimental results suggest that crack width is markedly less sensitive to concrete cover (Makhlouf and Malhas 1996). For this reason, the CEB-FIP model code (*Model* 1990) is used herein to predict the maximum crack width w_{max}, since crack width is relatively insensitive to cover and this crack width model has been calibrated using a larger amount of experimental data than most other crack width models (*Durable* 1992).

The CEB-FIP crack width model considers the influence of percentage of reinforcing steel, bond characteristics, bar size, concrete cover, concrete strength, steel stress, area of concrete around each bar stressed in tension, repeated loading, and shrinkage strain. Shrinkage strain is estimated from the B3 shrinkage model (Bazant 1995) and includes the effect of relative humidity h, effective cross section thickness, water-to-cement ratio and concrete strength f'_{cyl}. Statistical parameters for relative humidity and model error for the B3 model (ψ) are shown in Table 1. It is assumed herein that shrinkage is restrained (e.g., lack of contraction joints).

Most crack control provisions found in codes are developed from equations that predict the "probable maximum" crack width in a member, typically defined as the 90th-percentile value (Halvorsen 1987). However, a high degree of variability exists in crack widths with coefficients of variation estimated to be ~0.40 [e.g., Halvorsen (1987) and Nawy (1992)].

Corrosion initiation is not significantly affected by surface crack widths less than about 0.3–0.6 mm [e.g., Gergely (1981) and Wilkins and Lawrence (1983)]. Thus it is assumed herein that the critical threshold (surface) crack width is a uniformly distributed random variable within the range of 0.3–0.6 mm. The uniform distribution is taken to be appropriate for this case since there is no consensus on a characteristic value (i.e., mean). Subsequent sensitivity analyses have shown that this variable has a negligible influence on the results; therefore, this assumption is not critical. Note that a more rational critical threshold crack width might well be based on the crack width at the level of reinforcement.

CORROSION PROPAGATION

Corrosion of Reinforcement

It is assumed herein that the deterioration process caused by corrosion of reinforcement will lead to a uniform reduction in the bar diameter of the reinforcing steel $D(t)$; hence

$$D(t) = \begin{cases} D_i & t \le T_i \\ D_i - 2\lambda(t - T_i) & T_i < t \le T_i + (D_i/2\lambda) \\ 0 & t > T_i + (D_i/2\lambda) \end{cases} \quad (5)$$

where D_i = initial bar diameter; T_i = time to initiation; and λ = corrosion rate (at a surface) (mm/year). This assumption is the basis for most deterioration models, although it is recognized as probably being overly simplistic. The corrosion rate is most accurately measured from field/experimental studies as the current density i_{corr} (normally expressed in $\mu A/cm^2$), where $\lambda \approx 0.0116 i_{corr}$ (mm/year).

The corrosion rate will be governed by the availability of water and oxygen at the steel surface; this is likely a function of concrete quality, cover, and degree of cracking. Available information on the effects of such phenomena are limited and

TABLE 1. Statistical Parameters for Resistance and Loading Variables

Parameter (1)	Mean (2)	COV[a] (3)	Distribution (4)	Reference (5)
Depth[b]	D_{nom} + 0.8 mm	σ = 3.6 mm	Normal	Mirza and MacGregor (1979a)
C_{top}	$C_{t,nom}$ + 19.8 mm	σ = 16.5 mm	Normal	Mirza and MacGregor (1979a)
C_{bottom}	$C_{b,nom}$ + 8.6 mm	σ = 14.7 mm	Normal	Mirza and MacGregor (1979a)
f'_{cyl}	f'_c + 7.5 MPa	σ = 6 MPa	Lognormal	Attard and Stewart (in press, 1998)
k_w[c]	0.87	0.06	Normal	Stewart (1995)
$f'_{ct}(t)$	$0.69\sqrt{f'_c(t)}$	0.2	Normal	Mirza et al. (1979)
$E_c(t)$	$4{,}600\sqrt{f'_c(t)}$	0.12	Normal	Mirza et al. (1979)
f_{sy}	312 MPa	0.116	Beta	Mirza and MacGregor (1979b)
E_s	$1.005E_{s,nom}$	0.033	Normal	Mirza and MacGregor (1979b)
ψ (shrinkage)	1.0	0.34	Normal	Bazant (1995)
h	0.75	0.05	Normal	—
Model error	1.01	0.046	Normal	Ellingwood et al. (1980)
Dead load	1.05	0.10	Lognormal	Ellingwood et al. (1980)
Truck live load	287.5 kN	0.41	Normal	Nowak and Hong (1991)

[a]COV = coefficient of variation.
[b]From AASHTO data.
[c]k_w = workmanship factor (represents fair performance); $f'_c = k_w f'_{cyl}$.

J. Infrastruct. Syst. 1998.4:146-155.

somewhat contradictory (see next section). As such, the current density i_{corr} is taken as a uniformly distributed random variable within the range of 1–2 $\mu A/cm^2$; this is a "medium corrosion intensity" estimate (Dhir et al. 1994; Thoft-Christensen et al. 1996). It is assumed herein that the corrosion rate is constant once the critical threshold chloride concentration has been exceeded.

It is recognized that highly localized pitting of individual reinforcement bars is normally associated with chloride-induced corrosion. Given that pitting is spatially distributed, it is unlikely that many bars will be affected by pitting; hence, pitting corrosion will not significantly influence structural capacity at any given cross section. As such, the effects of pitting corrosion are not considered in the structural analysis.

Longitudinal Cracking and Spalling of Concrete Cover

The corrosion of reinforcement that transforms metallic ions to rust products may be accompanied by up to a 300% increase in volume [e.g., Mehta (1991)]. Consequently, the rust products exert tensile stresses in the concrete, causing longitudinal cracking (coincident cracks following the line of the reinforcement). Spalling of the concrete cover may then follow. It is therefore not surprising that spalling of concrete cover is a common problem in bridges, representing a hazard to passing vehicles and even to life safety (Broomfield 1997).

It has been suggested that reduced cover, longitudinal cracking, and spalling accelerate corrosion propagation [e.g., CS (1995)]. Yet somewhat surprisingly, experimental studies by Cabrera (1996) and Liu and Weyers (1996) both suggest that corrosion rate (as measured by i_{corr}) is relatively insensitive to longitudinal cracking and spalling. On the other hand, Cabrera (1996) and Schießl and Raupach (1997) have found that cover thickness has a significant influence on corrosion rate, while Liu (1996) found that corrosion rate appears not to be affected by cover depth. Although, it is most probable that corrosion rate is influenced by oxygen diffusion rate (cover thickness) and water content (concrete quality), it appears that no suitable quantitative (predictive) relationship between corrosion rate and cover (for exposure to deicing salts) is available in the existing literature. In light of the contradictory evidence and lack of predictive models, it is assumed simply herein that once corrosion is initiated, the corrosion rate is constant and is not affected by cover thickness, concrete quality or longitudinal cracking. This assumption is obviously nonconservative.

Transverse cracking will tend to result in highly localized corrosion and as such will not cause longitudinal cracking or spalling (Francois and Arliguie 1998). Conversely, chloride-induced (i.e., by chloride diffusion) corrosion will occur over a larger area (i.e., larger anode) and produce a more uniform buildup of corrosion products, thus resulting in longitudinal cracking and spalling. The model proposed by Liu and Weyers (1996) is used herein to predict the time to longitudinal cracking and spalling. The model is developed from that proposed by Bazant (1979a,b). Bazant's model determines time to cracking for chloride-induced corrosion based on a steady-state corrosion process. In the Liu and Weyers (1996) model, the time to cracking is influenced by corrosion rate, cover, bar spacing, concrete material properties, and creep coefficient. Liu and Weyers (1996) have modified Bazant's model by assuming that the rate of rust formation decreases as the rust layer grows since ionic diffusion is inversely proportional to the oxide distance. The model proposed by Liu and Weyers (1996) predicts times to longitudinal cracking and spalling that are in agreement with limited experimental data reported by Liu (1996). It should be recognized that existing longitudinal cracking models are subject to limitations, such as the large differences in volumes of different iron oxidations, predicting the kind and density of rust products and the assumption that corrosion products create equal pressure on the concrete. Further, existing models have either not been validated experimentally or have been validated only with very limited experimental results. This suggests that existing longitudinal crack models are probably of limited applicability. Statistical parameters assumed for all corrosion variables in this study are shown in Table 2.

TABLE 2. Statistical Parameters for Corrosion Variables

Parameter (1)	Mean (2)	COV* (3)	Distribution (4)
D	Eq. (3)	0.75	Lognormal
C_0	3.5 kg/m³	0.5	Lognormal
Cr	0.9 kg/m³	0.19	Uniform (0.6–1.2)
i_{corr}	1.5 $\mu A/cm^2$	0.33	Uniform (1–2)
w_{max}	1.0	0.4	Normal
Critical crack width	0.45 mm	0.19	Uniform (0.3–0.6)

*COV = coefficient of variation.

LOAD MODELS

The total bridge load is the sum of dead and truck live loads. Statistical parameters for dead and single truck loads are given in Table 1. It has been found that for multiple-lane bridges, the critical load effect occurs when two heavily loaded trucks are side-by-side and have fully correlated weights (Nowak 1993). If the number of crossings of two heavily loaded fully correlated trucks per year is N and the truck weight is normally distributed, it follows that the distribution of the weight of the heaviest truck (annually) is

$$F_n(w) = \left[\Phi\left(\frac{w - \mu_w}{\sigma_w}\right)\right]^N \tag{6}$$

where μ_w and σ_w = statistical moments of truck live load and $\Phi(\ )$ is the standard normal cumulative distribution function.

TIME-DEPENDENT RELIABILITY ANALYSIS

A corrosion-induced deterioration process will reduce the structural resistance of one or more structural elements. Hence the structural resistance is time-dependent and is denoted herein as $R(t)$. Further, structural loads may occur randomly in time and/or in intensity. If it is assumed that n independent load events S_j occur within the time interval $(0, t_L)$ at deterministic times t_j, $j = 1, 2, \ldots, n$, then the probability that the structure fails (i.e., exceeds strength limit state) anytime during this time interval is

$$p_f(t_L) = 1 - \Pr[(R(t_1) > S_1) \cap (R(t_2) > S_2) \cap \cdots \cap (R(t_n) > S_n)] \tag{7}$$

in which $t_1 < t_2 < \cdots < t_n$ [e.g., Kameda and Koike (1975)]. Clearly, the probability of failure is dependent upon the prior load and resistance histories, not simply on the extreme load up to time t_L and resistance at time t_L as is assumed in "time-invariant" reliability analyses, e.g., $p_f(t_L) = \Pr[\max(S_1, S_2, \ldots, S_n) > R(t_n)]$ (Fig. 3). Note also that both of these formulations are necessarily more complex than the simple time-independent analysis in which, for example, the resistance is constant over time, as is assumed in many structural reliability studies for code-calibration purposes. The analysis in this study is further complicated by (1) time-dependent load and resistance quantities that are not statistically independent (e.g., loads influence cracking, and cracking influences resistance); (2) flexural crack width being a function of prior load history (not just maximum load up to time t); (3) highly nonlinear

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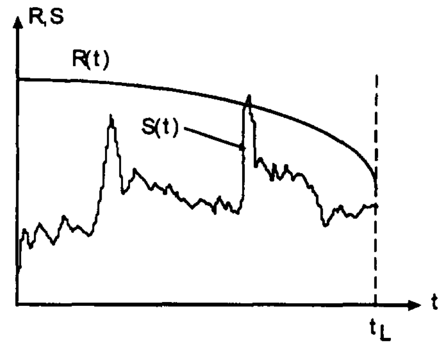

FIG. 3. Realizations of Load Effect *S*(*t*) and Resistance *R*(*t*)

limit-state functions; and (4) nonnormal random variables. Closed-form solutions currently cannot address all of these issues [especially (1) and (2)] and so are not tractable.

The other limit state considered herein is serviceability, namely, the onset of longitudinal cracking and spalling. In this case, the probability of spalling p_s is defined as

$$p_s(t) = \Pr(t > T_{cr}) \tag{8}$$

where T_{cr} = time to longitudinal cracking and spalling. The time to longitudinal cracking and spalling is influenced by a large range of time-dependent and strength-dependent variables as well as time to initiation (Liu and Weyers 1996). Given that (7) and (8) cannot be expressed in closed-form, Monte Carlo simulation is used herein to evaluate these limit-state probabilities.

ILLUSTRATIVE EXAMPLE—THREE-SPAN RC SLAB BRIDGE

Structural Configuration and Assumptions

The bridge considered in this study is a 28-year-old three-span continuous RC slab bridge (Huria et al. 1993) (Fig. 4). The deck slab is 11 m wide with a design thickness of 406 mm, including a 19-mm integrated wearing course. It is assumed here that the top 5 mm of the slab are grooved and that only the remaining wearing course contributes to the structural capacity of the slab. The top and bottom reinforcement ratios over the supports are 1.38% (D_i = 32.3 mm, #10) and 0.56% (D_i = 28.7 mm, #9), respectively. The top and bottom reinforcement ratios at midspan are 0.12% (D_i = 19.1 mm, #6) and 1.12% (D_i = 28.7 mm, #9), respectively. Reinforcing steel is Grade 40 (276 MPa), specified concrete strength f'_c is 4,000 psi (27.6 MPa), and it is assumed that top and bottom clear covers are 50 and 25 mm, respectively.

The statistical parameters for dimensions, materials, and loads appropriate for the three-span continuous RC bridge are given in Table 1. It is assumed that (1) the number of fully correlated side-by-side trucks (N) per year for this bridge is 600 (Nowak 1993); (2) only the slab deck is exposed to deicing salts; (3) slab depth, all material properties (f'_c, f'_{ct}, f_{sy}, E_c, E_s) and all corrosion parameters (D, C_0, Cr, i_{corr}) are constant across the entire bridge; (4) reinforcement covers and areas of reinforcing steel are statistically independent at each cross section; and (5) concrete compressive strength is time dependent such that

$$f'_c(t) = \frac{t}{\gamma + \omega t} f'_c(28) \tag{9}$$

where t is in days; $\gamma = 4.0$ and $\omega = 0.85$ for moist cured normal (Type I) portland cement ("Prediction" 1978). In reality, if a low clearance bridge passes over traffic lanes then it is likely that its slab soffit would also be exposed to chlorides from car spray. This is an area for future consideration.

The analysis assumes statistically independent annual maximum truck loads; hence time-dependent failure probabilities can be calculated from (7) and (8) considering annual maximum loads (two heavily loaded side-by-side trucks) and the cracking/damage they cause. Failure probabilities are calculated for 100 successive annual time increments. Flexural actions are computed for (1) middle of central span, i.e., "midspan"; and (2) over the supports. Failure is deemed to occur if the flexural actions exceed the moment capacity (structural resistance) at either of these cross sections. Axle spacings and distribution of axle loads are based on the standard HS-20 truck and were used to calculate peak flexural actions at these cross sections.

Results

Structural (Strength) Failure

The mean structural resistance over the support as a function of time is shown in Fig. 5. In the present case, it is observed that corrosion can cause up to a 13% decrease in mean flexural resistance of a structural section. Note that deicing salts tend to corrode the top reinforcing bars (assuming no leaks exist in construction joints, etc.), leading to structural failure only over the supports, i.e., location of maximum negative moments. The influence of this decrease in resistance is evident from Fig. 6 that shows the resulting probability of structural failure obtained from the Monte Carlo simulation analysis. For comparative purposes, the failure probabilities for the ideal case of no deterioration (i.e., time invariant resistance) is shown also. It is observed that deicing salts lead to a large loss of structural safety (failure probability increases by more than two orders of magnitude after 80 years). This observation is in broad agreement with field data of bridge performance.

The reliability of a structure, given that it has survived for T years, will increase for service proven (older) structures, i.e., as survival age T increases. Prior service loads may result in the failure of some undersized elements; thus the lower tail of the distribution of structural resistance will be progressively truncated as survival age increases. This revised time-dependent distribution of structural resistance can then be used to calculate an updated estimate of structural reliability [see Stewart (1997) for further details]. Thus, information about prior satisfactory structural performance can be used to update estimates of time-dependent bridge reliability. The probability that the bridge will fail (in flexure) in t subsequent years given that it has survived T years of service loads is denoted as $p_f(t|T)$ and is shown in Fig. 7 for the cases of deterioration (deicing salts) and no deterioration. It is observed from Fig. 7 that the probability of structural failure reduces for older

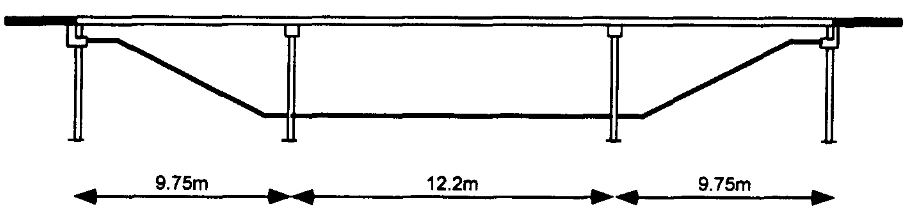

FIG. 4. Three-Span RC Slab Bridge

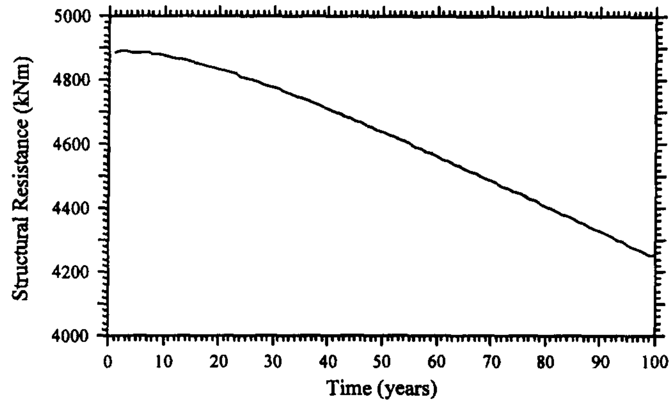

FIG. 5. Mean Structural Resistance over Support

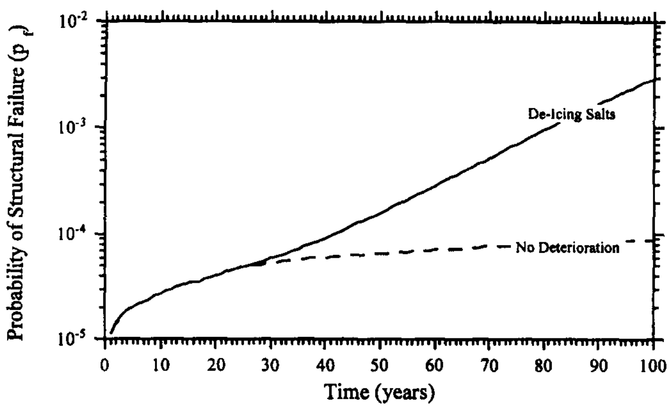

FIG. 6. Time-Dependent Probability of Structural Failure (Strength Limit State)

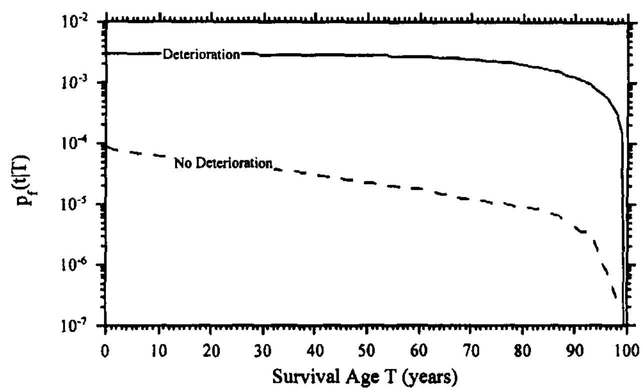

FIG. 7. Time-Dependent Probability of Structural Failure for a Service-Proven Structure

(proven) structures. Not surprisingly, probabilities of failure are greater for a bridge subject to deterioration.

Spalling

Fig. 8 shows the probability of spalling for deterioration caused by deicing salts, again obtained using Monte Carlo simulation. The mean time to spalling is in the region of 30–40 years (Table 3). However, spalling can occur much earlier; for example, there is a 5% chance that spalling will occur within the first 5 years. Proportionally, 35% of spalling first occurred over the supports and 65% first occurred at midspan. Spalling was more likely to occur at midspan since the diameter of the reinforcing bar is smaller (effect of corrosion proportionally greater), and hence the subsequent time to spalling (given that corrosion had been initiated) is less at midspan (Table 3).

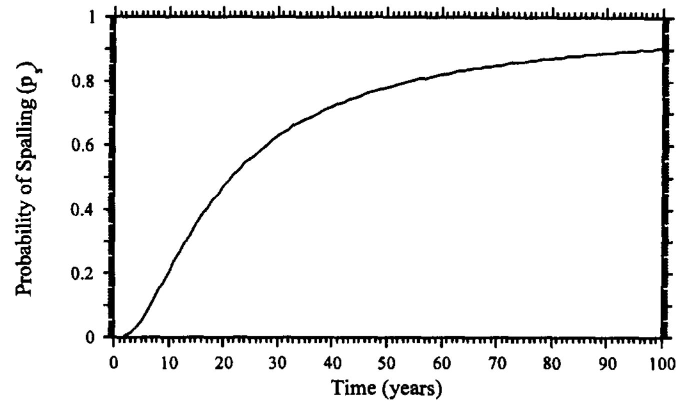

FIG. 8. Time-Dependent Probability of Spalling

TABLE 3. Average Time to Spalling (Years)

(1)	Support (2)	Midspan (3)	Minimum (4)
Subsequent time to spalling given that corrosion has been initiated	16.8	7.1	—
Total time to spalling	38.3	30.8	27.0

Interaction between Spalling and Structural Failure

Serviceability limit states such as longitudinal cracking and spalling may often be precursors to more critical and dangerous strength limit-state problems. As well as serving as "warnings" of perhaps more serious problems to follow, nonserviceability can further exacerbate (i.e., accelerate the onset of) strength limit-state failures. For example, it may be of interest given that unserviceability has occurred (i.e., spalling) to evaluate an updated probability of structural failure (collapse). This can be expressed as a conditional probability of structural failure; namely, the probability that the structure will fail (collapse) between T and 100 years given that spalling has just occurred at T years.

Fig. 9 shows this conditional probability of structural failure for spalling over the support (i.e., rutting) if it is assumed that spalling or low concrete cover increases corrosion rate by, for instance, a factor of 2. As expected, the conditional probability of structural failure decreases as the time of detection of first spalling increases. Note that corrosion initiation and spalling will normally occur at an early age only if the cover is very low. Other trends could be obtained for other strength and serviceability limit states such as shear or serviceability (deflection) where loss of reinforcement area or reduction of bond would also be critical to structural performance. Nonetheless, the present example shows the potential for using known in-

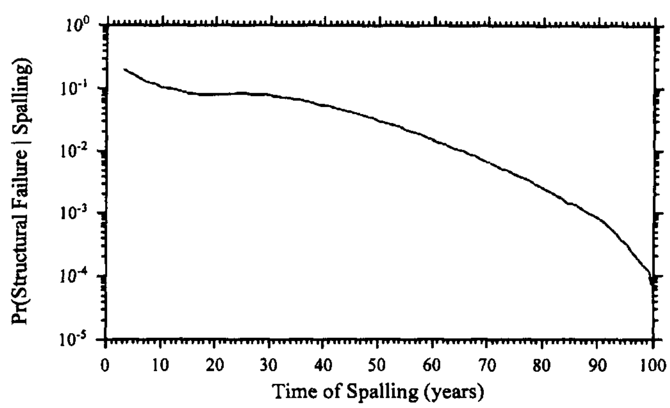

FIG. 9. Probability that Structure Will Fail after *T* Years Given that Spalling Occurs at *T* Years

formation about the exceedance of serviceability limit state to update the probability of structural failure.

Causes of Corrosion Initiation

Fig. 10 shows the probability density functions for diffusion and transverse cracking-induced initiation for the top reinforcing steel over the support. For the slab surfaces of the bridge considered herein, penetration of chlorides by diffusion is the dominant cause of corrosion initiation. Transverse cracking that exceeds the critical threshold crack width occurs early in the lifetime of the bridge, although its incidence is low, and so it is seldom the cause of initiation. It is interesting to note that when Stewart and Rosowsky (1998b) used a different cracking model BS 8110 (or ACI 318) transverse cracking was the dominant cause of corrosion initiation for the top reinforcement over the support. This was due mainly to the characteristic of the flexural cracking model (BS 8110, ACI 318) to significantly increase crack width as cover increases, a phenomenon that has not been observed in laboratory or field tests. As described previously, the transverse crack model used in the present paper is deemed to be more realistic than that used by Stewart and Rosowsky (1998b). Shrinkage accounted for ~20% of total transverse crack width, and so it is not considered a major cause of corrosion initiation.

Influence of Concrete Compressive Strength and Concrete Cover

The influences of cover and specified concrete compressive strength on resistance and spalling failure probabilities were also evaluated using time-dependent probabilistic analysis. To isolate cover and specified concrete compressive strength as the main variables, only the cross section over the supports was considered, and the probability of failure is therefore the probability that the actions (bending moments) exceed the moment capacity at this support. Fig. 11 shows how cover and specified concrete compressive strength influence the mean structural resistance. To maintain the structural resistance at a similar level the reinforcement area was adjusted as cover and specified concrete compressive strength varied. It is evident from Figs. 12 and 13 that reduced cover and/or specified concrete compressive strength lead to an increase in structural failure and spalling probabilities, respectively. Further details and additional findings are presented elsewhere (Stewart and Rosowsky 1998a).

The results from the analysis suggest that structural reliabilities are not significantly affected by cover in the range of 25–100 mm (ACI 318 specifies a minimum cover of 50 mm). It might be expected that reduction in failure probabilities as cover increases would be more pronounced than that shown

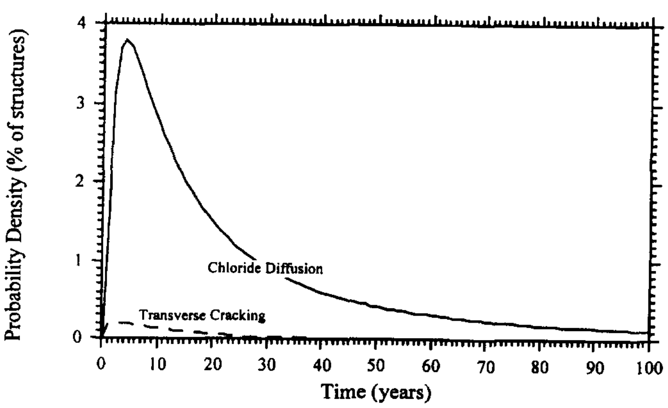

FIG. 10. Probability Distributions for Diffusion and Cracking-Induced Initiation—Top Reinforcement over Support

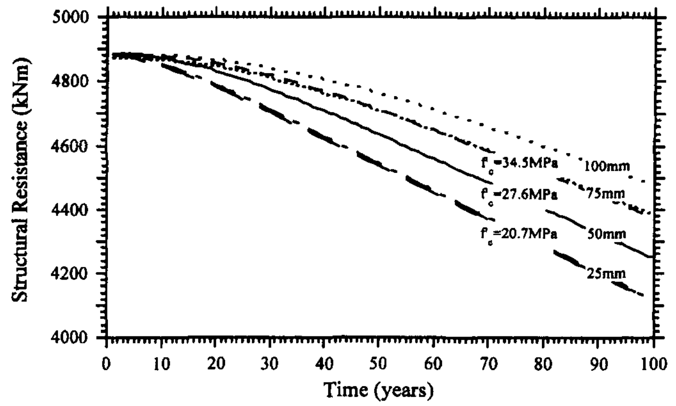

FIG. 11. Influence of Cover and Concrete Compressive Strength on Structural Resistance

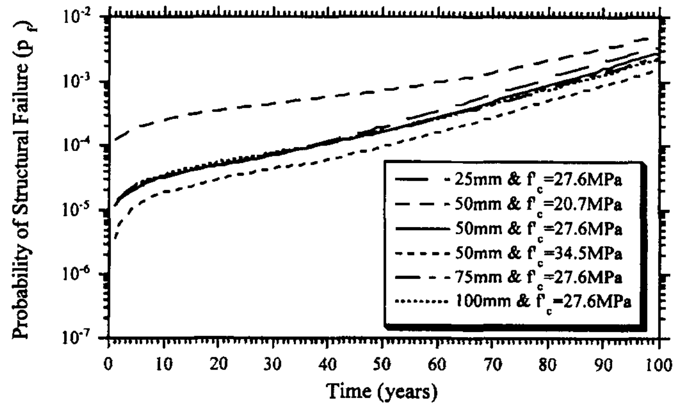

FIG. 12. Influence of Cover and Concrete Compressive Strength on Probability of Structural Failure (Strength Limit State)

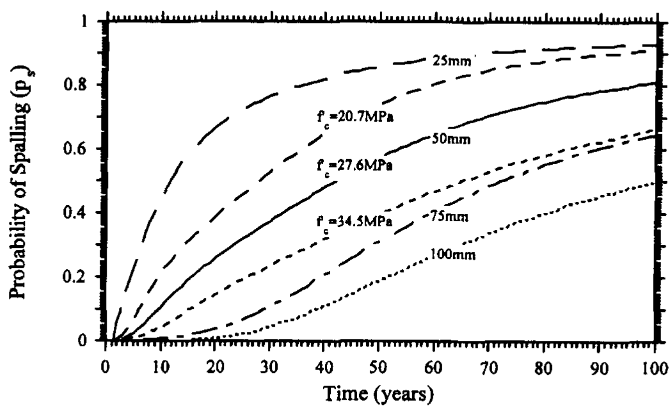

FIG. 13. Influence of Cover and Concrete Compressive Strength on Probability of Spalling

in Fig. 12. While increasing cover further impedes the diffusion of chlorides, existing crack width models suggest increasing cover also leads to wider surface cracks. On the other hand, reduced cover and/or specified concrete compressive strength lead to a significant increase in probability of spalling. Note that the coefficient of variation in resistance increases with specified concrete compressive strength (Table 1); hence the probability of structural failure at $t = 0$ is higher for elements with low specified concrete compressive strengths. The results from the present analysis suggest that reduction in specified concrete compressive strength and reduced cover are both significant causes of degradation in structural and serviceability performance.

Sensitivity Analysis

A sensitivity analysis was conducted to ascertain the effect of cover, concrete strength, and corrosion model parameter uncertainty on (1) structural resistance after 100 years; and (2) time to spalling. In addition to providing information on the relative importance of the parameters, the findings from such an analysis can be used to (1) develop appropriate quality assurance measures; and (2) prioritize resources for describing more accurately the behavior of these variables. The corrosion rate i_{corr} and cover were found to significantly affect structural resistance (and thus the failure probabilities). Not surprisingly, cover, diffusion coefficient, surface chloride content, corrosion rate, and critical threshold chloride concentration Cr are all significant parameters for predicting time to spalling. Complete details and results from the sensitivity analysis are presented elsewhere (Stewart and Rosowsky 1998a). It should be noted that results of sensitivity analyses may differ considerably for other bridge configurations. For example, bridges with a thicker cover are more likely to be sensitive to cracking rather than diffusion parameters.

Analysis

The present analysis (1) ignores system effects such as collapse mechanisms and load redistribution; (2) assumes that all reinforcement corrodes uniformly across the entire bridge deck; (3) assumes that corrosion will occur only at the midspan and over the support; and (4) assumes the absence of inspections and repairs (hence failure occurs even though signs of structural distress might indicate a need for repair); these necessarily lead to the overestimation of failure probabilities. Further, the results presented herein are subject to uncertainty because of the inadequacy of existing models for corrosion initiation and propagation. It is clear that much scope exists for further work.

The failure probabilities calculated herein should be considered "notional" and therefore the results should be used for comparative purposes only. Also, the reliabilities computed herein are for a typical slab-type RC bridge with parameters reflecting a variety of construction practices, environmental conditions, etc. A reliability analysis can also be conducted for a specific bridge. In such cases the coefficient of variation for some parameters (e.g., D, C_0, i_{corr}) would most likely be lower than the values used in the present study (i.e., increased certainty), and in some cases the mean values may also be different. In general, this will result in a calculated probability of failure lower than that obtained in this study.

CONCLUSIONS

A probabilistic framework has been developed for the time-dependent analysis of RC bridge decks and the consequent loss of structural and serviceability performance due to chloride-induced corrosion. Corrosion initiation and propagation reduces the cross-sectional area of the steel reinforcing bars, leading to a loss in structural strength, while corrosion products may cause longitudinal cracking and spalling. The influence of deicing salts, flexural, and shrinkage cracking and critical thresholds (chloride concentration, crack width) have been incorporated into the analysis. A time-dependent reliability analysis considering both ultimate strength and spalling limit states has been developed using the described deterioration models. A lifetime reliability analysis of a three-span RC slab bridge found that the application of deicing salts causes significant long-term deterioration and reduction in structural safety; this observation is in agreement with field data of bridge performance. Concrete cover and specified concrete compressive strength were found to have the greatest influence on the probability of spalling.

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