

Assessing the Performance of Corroding RC Bridge Decks: A Critical Review of Corrosion Propagation Models

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ABSTRACT: Corrosion of steel reinforcement is one of the most prevalent causes of reinforced concrete (RC) structures deterioration in chloride-contaminated environments. As a result, evaluating the impact of any possible corrosion-induced damages to reinforced concrete bridges strongly affects management decisions: such as inspection, maintenance and repair actions. The corrosion propagation phase is a significant factor in the service life of reinforced concrete structures and thus, it requires appropriate attention. Various models have been developed to simulate and/or predict the propagation phase. This paper proposes that a method in which a reliability framework is developed to assess the durability of reinforced concrete structures under corrosion in marine environments functions better. The main concern of this study is chloride-induced corrosion. In due regard, serviceability and ultimate limit states of the structure are also taken into account. Subsequently, the proposed method is employed to review the existing models for prediction of corrosion propagation.

Keywords: Corrosion, Deterioration, Propagation Models, Reinforced Concrete Bridges, Structural Reliability.

INTRODUCTION

Lately, the corrosion deterioration of reinforced concrete structures has drawn considerable attention to itself. Researchers are studying this phenomenon analytically and/or experimentally worldwide. Durability and serviceability of corroded reinforced concrete structures have been investigated to determine the relationship between the corrosion process and the service life of reinforced concrete structures. These relationships have been used to develop

service life models. However, there are still many aspects which need further extensive research; such as predicting the real long-term effects of corrosion on steel reinforcement and the long-term reliability of concrete structures.

According to the well-known general corrosion model, developed by Tutti (1982), service life of reinforced concrete structures subject to corrosion is comprised of two general phases: initiation and propagation. Initiation phase is the depassivation process of reinforcement, when the aggressive

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agents are transported into the concrete and reach to the steel reinforcement surface. Propagation phase begins when the steel is depassivated, causing active corrosion, and terminates when reinforced concrete structure reaches the end of its service life. Significant efforts have been made in modeling the initiation phase. Much less efforts have been focused on the propagation phase. Hence, this paper focuses on the propagation phase of the deterioration process.

Due to the complex nature of corrosion process, different types of propagation models have been developed for corrosion. In almost all of these modeling, corrosion current has been regarded as the measure of deterioration rate. Current linear polarization method has successfully been used for measuring the corrosion rate in reinforced concrete structures. But the corrosion rate obtained via this method is only an instantaneous value corresponding to a certain concrete temperature and moisture content at measurement moment (Liu and Weyers, 1998). To accurately predict corrosion rate, it is necessary to predict both the rate and the severity of damage reliably, and to plan for the maintenance of these structures. However, researchers have failed to attach due importance to the prediction of corrosion. The direct consequence of this may be under- or over-estimation of both the severity of damage and the remaining time until corrosion-induced damage sets in; such as loss of steel reinforcement, and the consequent termination of the structure's service life. This makes the task of selecting an appropriate model difficult for the practicing engineer.

In this study, a structural deterioration reliability (probabilistic) framework is developed for reinforced concrete bridge decks exposed to marine environments. Henceforth, serviceability and ultimate limit states are of central concern. Besides, this

method is used to study some existing models with the aim of predicting propagation phase of the corrosion process.

MATERIALS AND METHODS

Corrosion Initiation

Chloride penetration is a complex phenomenon that depends on several key mechanisms, such as capillary movement, diffusion, absorption, etc. Models based on diffusion theory have been developed to represent the chloride ingress in concrete and are widely used in practice to predict the initiation of reinforcement corrosion in concrete structures (Andrade et al., 1996). Diffusion is mathematically represented by the partial differential equation using Fick's second law of diffusion. The model used for chloride ingress is as follows (Tutti, 1982):

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

where C_0 is the surface chloride concentration; D is the effective diffusion coefficient; x is the depth at which chloride concentration is measured; t is the time of exposure; $C(x, t)$ is the chloride concentration at depth x and time t .

Re-arranging the above equation and replacing $C(x, t)$ with the critical threshold chloride concentration for corrosion initiation (C_{th}) leads to:

$$T_{corr} = \frac{E_{mod} x^2}{4D \left[\operatorname{erfc}^{-1} \left(\frac{C_{th}}{C_0} \right) \right]^2} \quad (2)$$

where T_{corr} is the time to corrosion initiation at any depth (x) from the surface.

Corrosion Parameters

Unfortunately, in-site surveys of surface chloride concentration in concrete structures exposed to marine environments are both

very limited and scattered. Due to the absence of in-site data, surface chloride concentration can be estimated by empirical based formulas reported by McGee (1999). McGee, according to a field-based study suggested that the surface chloride concentration as a function of the distance from the coast (d in km) can be deduced accordingly:

$$\begin{aligned} C_0(d) &= 2.95 \text{ kg/m}^3 \\ d &< 0.1 \text{ km} \\ C_0(d) &= 1.15 - 1.81 \log_{10}(d) \text{ kg/m}^3 \\ 0.1 \text{ km} &< d < 2.84 \text{ km} \\ C_0(d) &= 0.03 \text{ kg/m}^3 \\ d &> 2.84 \text{ km} \end{aligned} \quad (3)$$

McGee found that for coastal distances exceeding 0.1 km from the ocean, the coefficient of variation for surface chloride concentration is 0.49. For the present study, a coefficient of variation of 0.5 is used.

The chloride diffusion coefficient (D_c) represents concrete permeability and is estimated by the model developed by Papadakis et al. (1996):

$$D_c = D_{H_2O} 0.15 \frac{1 + \rho_c \frac{c}{w}}{1 + \rho_c \frac{w}{c} + \frac{\rho_c a}{\rho_a c}} \left(\frac{\rho_c \frac{w}{c} 0.85}{1 + \rho_c \frac{w}{c}} \right)^5 \text{ (cm}^2/\text{s)} \quad (4)$$

where a/c is the aggregate to cement ratio, w/c is water to cement ratio, ρ_c and ρ_a are the mass densities of cement and aggregates, respectively and D_{H_2O} is the chloride diffusion coefficient in an infinite solution ($=1.6 \times 10^{-5} \text{ cm}^2/\text{s}$ for NaCl).

Literature review reveals a large number of data obtained from chloride threshold concentration. However, there is little agreement among the measured values estimated. In this paper, the statistical parameters for critical threshold chloride concentration are the same as those reported by Stewart and Rosowsky (1998).

Corrosion Propagation

Various models have been proposed for predicting the propagation of corrosion. A brief review of some available corrosion propagation predicting models is presented as follows.

Vu and Stewart's Model: Based on this assumption that the average relative humidity for many locations in Australia, the United States, Europe and Asia is over 70%, Vu and Stewart (2000) suggested that the corrosion rate was limited by the availability of oxygen on the steel surface. Oxygen availability depends on concrete quality (w/c ratio), cover and environmental conditions (temperature and relative humidity). They proposed that the influence of w/c ratio and cover may be expressed empirically as follows:

$$i_{corr}(1) = \frac{\alpha(1 - w/c)^{-1.64}}{C} \text{ (}\mu\text{A/cm}^2\text{)} \quad (5)$$

where $i_{corr}(1)$ is the corrosion rate at the onset of corrosion propagation; C is the concrete cover in mm, and α is a parameter that depends on environmental conditions. Suo and Stewart (2009) suggested that for an ambient relative humidity of 80%, $\alpha=27$ and for an ambient relative humidity of 75% and temperature of 20 °C, corrosion rate is 37.8.

They used data reported by Liu and Weyers (1998) to develop a relationship between time and the corrosion rate, which is expressed empirically as:

$$i_{corr}(t_p) = i_{corr}(1) \cdot 0.85 \cdot t_p^{-0.29} \quad (6)$$

where t_p is the corrosion propagation time.

This model is extensively used in relevant literature (e.g. Duprat, 2007; Firouzi and Rahai, 2011; Suo and Stewart, 2009 and Stewart and Suo, 2009).

Li and Lawanwisut's Model: Li and Lawanwisut (2003), based on the

experimental data reported by Li (2002), suggested that the corrosion rate increases logarithmically over time, estimable from the following equation:

$$i_{corr} = 0.3683 \ln(t) + 1.1305(\mu A/cm^2) \quad (7)$$

Experimental data obtained under a simulated marine condition for flexural members.

Yalcyn and Ergun's Model: The model by Yalcyn and Ergun (1996) was developed by studying the effect of chloride and acetate ions on i_{corr} . Corrosion was evaluated by measuring half-cell potentials (HCP) and LPR. The model was developed based on results obtained from accelerated corrosion testing (admixed chlorides); with i_{corr} measurements taken up to a period of 90 days i.e. at 1, 7, 28, 60 and 90 days on cylindrical specimens of 150 mm in diameter and 150 mm high. They suggested that corrosion rate decreases exponentially as time progresses according to the subsequent equation:

$$i_{corr}(t) = i_{corr,0} \cdot e^{(-ct)} (\mu A/cm^2) \quad (8)$$

where $i_{corr,0}$ is the initial corrosion rate and c is a corrosion constant that depends on the structure and the properties of concrete, namely concrete pore saturation degree, pH, permeability and the cover thickness of the concrete. Yalcyn and Ergun proposed a value of c (evaluated from i_{corr} vs. time curves) as $1.1 \times 10^{-3} \text{ day}^{-1}$ for the different concrete samples they considered.

Ahmad and Bhattacharjee's Model: An empirical model has been developed by Ahmad and Bhattacharjee (2000) for chloride induced reinforcement corrosion of rebar in concrete under normal exposure. The main factors considered are w/c-ratio, cement content and the chloride content of the concrete. In order to evaluate the

simultaneous effects of these factors on rebar corrosion in terms of corrosion indicators, such as half-cell potential, concrete resistivity, corrosion rate, free chloride content and pH of the concrete, a standard statistical experiment design has been adopted. Through analysis of variance (ANOVA), the factors and their possible interactions affecting each corrosion indicator have been identified. After identifying the effect of the factors and their possible interactions on each of the corrosion indicators separately, the empirical models for corrosion indicators have been fitted in terms of the effective factors and interactions, using the method of least squares (Raupach, 2006):

$$i_{corr} = 37.726 + 6.12 \cdot C \cdot 2.231 \cdot A^2 \cdot B + 2.722 \cdot B^2 \cdot C^2 (nA/cm^2)$$

$$A = \frac{\text{Cement Content}(kg/m^3) - 300}{50}$$

$$B = \frac{w/c - 0.65}{0.075}$$

$$C = \frac{\text{CaCl}_2(\text{by the weight of cement}) - 2.5}{1.25}$$
(9)

Breysse et al.'s Model: Breysse et al. (2008) proposed an empirical relationship for corrosion current by multi-linear regression of large laboratory and field measurements:

$$\ln(i_{corr}) = 0.0312RH - 4736/T + 1.695w/c - 0.391C + 14.589 \quad (10)$$

where RH is the air relative humidity (%), T is the air temperature (K), w/c is the water to cement ratio and C is the concrete cover (cm). The corrosion current density is

obtained in ($\mu A/cm^2$) (El Hassan et al., 2010).

Li's Model: The models discussed previously used corrosion current as a measure for deterioration rate. Li (2003) proposed a different procedure to model deterioration of structural resistance. He considered the randomness of structural deterioration and its time-variant nature, and suggested to model structural deterioration of structures as a stochastic process, quantified by a deterioration function as follows:

$$\varphi(t) = \frac{R(t)}{R_0} \leq 100\% \quad (11)$$

where $\varphi(t)$ is the deterioration function, $R(t)$ is the structural resistance at any given time t and R_0 is the original structural resistance.

Li modeled the deterioration function based on experimental results, by using a mean function, $\mu_\varphi(t)$ and a function of coefficient of variation, $V_\varphi(t)$ in the ensuing equation:

$$\begin{aligned} \mu_\varphi(t) &= \exp(-0.027t) \\ V_\varphi(t) &= 0.016t \end{aligned} \quad (12)$$

Corrosion Cracking

Accumulation of the corrosion products exert internal tensile stresses upon the concrete, which leads to the appearance of cracks in the concrete structures. The appearance of the first corrosion-induced crack is usually regarded as the end of the functional service life, where rehabilitation of the corroding structural element is required (Weyers, 1998). In this paper, the mathematical model developed by El Maaddawy and Souki (2007) is used in order to predict the time span from corrosion initiation to corrosion cracking. They suggested that the internal radial pressure

generated by corrosion can be expressed as follows:

$$P_{corr} = \frac{m_1 E_{ef} D}{90.0(1 + \nu + \psi)(D + 2\delta_0) - 2\delta_0 E_{ef}} \quad (13)$$

$$(1 + \nu + \psi)(D + 2\delta_0)$$

where D is the original diameter of the steel reinforcing bar, E_{ef} is the effective elastic modulus of concrete that is equal to $[E_c / (1 + \phi_{cr})]$, E_c is the elastic modulus of concrete, ϕ_{cr} is the concrete creep coefficient (2.35 per ACI 209R-5), ν is the Poisson's ratio of concrete (0.18), δ_0 is the thickness of the porous zone and it is typically in the range of 0.01–0.02 mm (Thoft-Christensen, 2000) and $\psi = (D + 2\delta_0)^2 / 2C(C + D + 2\delta_0)$.

The percentage of steel mass loss, m_l , is estimated by:

$$m_l = 100 \left(\frac{M_{loss}}{M_{st}} \right) \quad (14)$$

where M_{loss} is the mass of steel per unit length consumed for rust to generate, and M_{st} is the original mass of steel per unit length before the inception of corrosion damage.

The radial pressure required for concrete cover to crack, P_{cr} , is assessed by:

$$P_{cr} = \frac{2Cf_{ct}}{D} \quad (15)$$

where f_{ct} is the concrete tensile strength and C is concrete cover.

Time Variant Load Model

For one-lane short span bridges, the critical loading effects generally occur when heavily loaded trucks cross the bridge. Past experiences suggest that traffic loads and volume increases continuously. If (i) annual increases in truck loads and heavy traffic

(truck) volume are represented by λ_m and λ_v , respectively; (ii) number of heavily loaded trucks crossing the bridge per year is N , and (iii) truck weight is normally distributed, then the annual time-variant cumulative distribution of the weight of the heaviest truck w will be:

$$F_n(w, t) = \left[\phi \left(\frac{w - \mu_w * (1 + \lambda_m)^t}{\sigma_w * (1 + \lambda_m)^t} \right) \right]^{N * (1 + \lambda_v)^t} \quad (16)$$

where t is time in years, σ_w and μ_w are statistical parameters of live load for a single truck and ϕ is the cumulative function of the standard normal distribution (Vu and Stewart, 2000). Maximum annual truck loads are supposed to be statistically independent. In this paper, λ_m and λ_v are assumed to be 0.5% and 2.3%, respectively (Vu and Stewart, 2000).

Probabilistic Lifetime Evaluation and Reliability Analysis

Due to uncertainties in material properties, environmental conditions and corrosion model parameters, it is not possible to predict the lifetime of RC structures accurately. For this reason, the reliability analysis is mandatory if we are to obtain practical information on the effects of each parameter and the characteristics of the predictive models. In order to have a probability-based lifetime evaluation of bridge decks, serviceability and ultimate limit states are defined as follows:

Serviceability limit state: In this paper, the appearance of the first corrosion crack is identified as serviceability limit state, which is calculated by the following equation:

$$L_s(X, t) = P_{cr} - P_{corr} \quad (17)$$

where P_{cr} is the radial pressure required to cause cracking, P_{corr} is the internal radial

pressure caused by corrosion, X is the vector of basic variables and t is the age of the structure. $L_s > 0$ denotes no cracking and $L_s < 0$ indicates that the concrete is cracked.

Ultimate limit state: Structural integrity is mainly dependent on the ultimate flexural capacity (shear failure is not considered herein). The ultimate limit state function, L_u , is considered as a function of all the basic variables, X . It can be estimated at a certain time, t , within the intended service period using:

$$L_u(X, t) = \text{Capacity} - \text{Demand} = R - S \quad (18)$$

The RC member fails when the safety margin plunges to zero; i.e. when the applied moment (load effects) reaches the flexural capacity of the member.

Flexural capacity of the RC bridge deck at a critical section, R , is calculated from ACI318-02:

$$R = A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c b} \right) \quad (19)$$

where f_y is the yield strength of reinforcing steel, A_s is the total area of steel in tension zone, d is effective depth and b is the width of the concrete section. The changes in the capacity of the structure are due solely to the loss of cross-sectional area of the reinforcing steel.

A time-dependent evaluation of the load effects, S , is also taken into account. Although the load effects owing to dead load are assumed constant, the maximum live load is assumed to increase over time. The live load model was discussed previously.

The probabilistic evaluation of the serviceability and ultimate limit states of the deck slab are conducted using Monte Carlo simulation. Monte Carlo simulation involves "sampling" at "random" to simulate artificially a large number of experiments

and to observe the results (Melchers, 1999). In this paper, Latin Hypercube Sampling (LHS) method is used to increase the efficiency (required number of the samples and hence the computation time, computer resources, and etc.) of the simulation process (Olsson et al., 2003).

A computer code is compiled to generate the random variable inputs needed for the Monte Carlo simulation and reliability analysis. The computer code is written in MATLAB ® version R2010a. Figure1 is a flowchart describing the algorithm used.

Application Example

As an application example, a virtual reinforced concrete bridge deck exposed to corrosion is considered. This bridge is a simply supported reinforced concrete slab bridge with a span length of 7 m and a width of 3.65 m. The bridge was designed according to the ACI318-02 building code for the HS20 loading. Statistical parameters for dimensions, material properties and loads for this reinforced concrete bridge are given in Table 1.

Table 1. Statistical parameters for resistance and loading variables.

Parameter	Value or Mean	COV	Distribution
Surface chloride concentration, C_0	Eq.(3)	0.5	Lognormal
Threshold chloride concentration, C_{th}	0.9 kg/m ³	0.19	Uniform
Diffusion coefficient, D_c	Eq.(4)	-	-
Water to cement ratio, w/c	0.45	0.05	Normal
Cement content	355 kg/m ³	0.05	Normal
$CaCl_2$	0.5%(by the weight of cement)	-	Deterministic
Model error (T)	1.0	0.2	Lognormal
Model error (D_c)	1.0	0.2	Normal
Thickness of the porous zone, δ_0	-	-	Uniform(0.01-0.02 mm)
Cover depth	50 mm	0.1	Normal
Asphalt depth	100 mm	0.1	Normal
Compressive strength of concrete, f'_c	250 kg/m ²	0.15	Normal
Yield strength of reinforcing steel, f_y	4000 kg/m ²	0.1	Normal
Concrete tensile strength, f_{ct}	*0.53 (f'_c) ^{0.5}	-	-
Concrete elastic modulus, E_c	*0.043w _c ^{1.5} (f'_c) ^{0.5}	-	-
Concrete unit mass, w_c	2.4 ton/m ³	-	Deterministic
Asphalt unit mass	2.2 ton/m ³	-	Deterministic
Single truck live load(heavily loaded)	30 ton	0.15	Normal

* f'_c is in MPa and w_c is in kg/m³

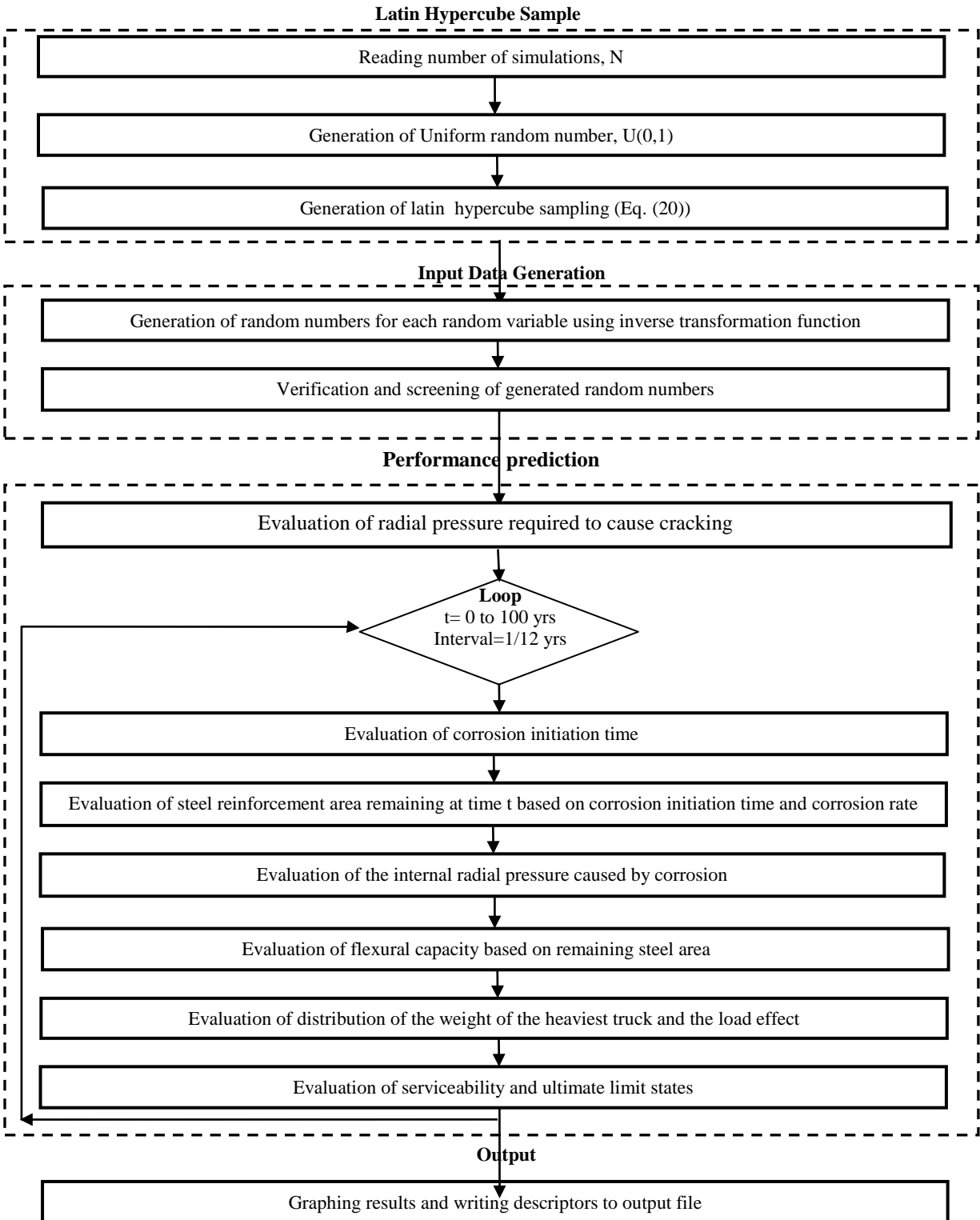


Fig. 1. Flowchart of the reliability model.

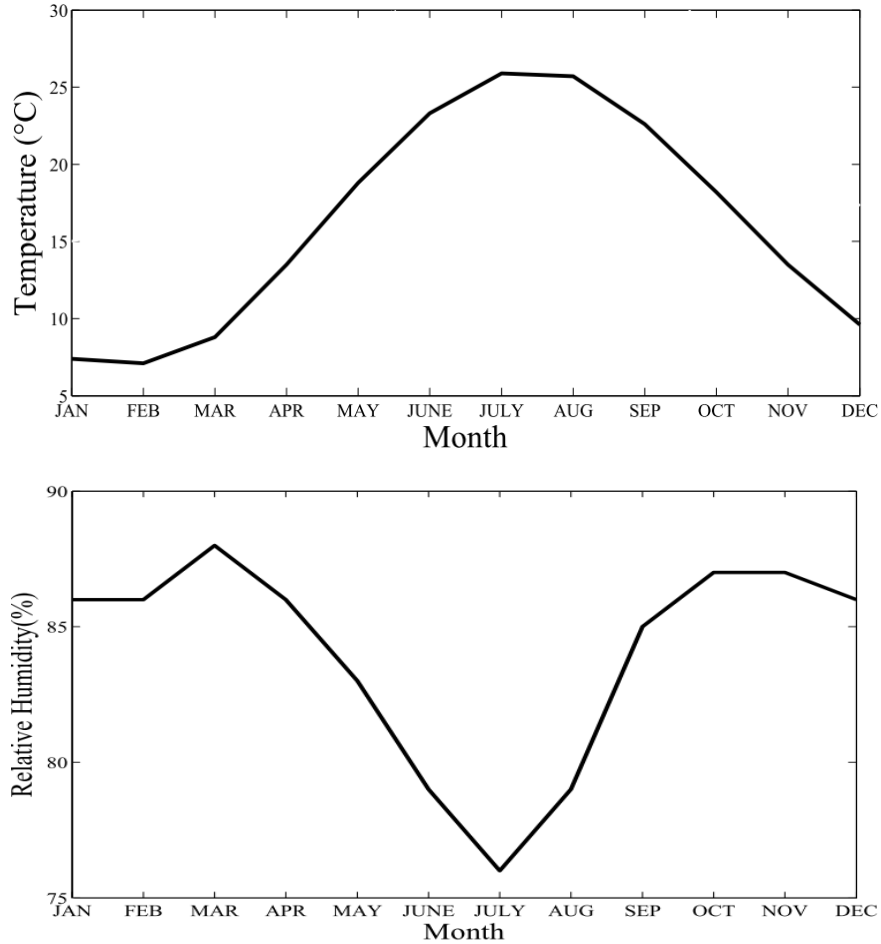


Fig. 2. Climatic data from meteorological stations of Anzali Port.

The bridge is considered to be located in Bandar-e Anzali (Anzali Port). Bandar-e Anzali, the most humid city in Iran, is a harbor town located on the southern coast of Caspian Sea. Figure 2 represent real climatic data obtained from meteorological stations of the concerned region. The monthly average values of temperature and relative humidity are calculated by averaging the monthly values of each parameter for 55 years (1951–2005). The bridge distance from the coast is 0.15 km.

RESULTS

In this study, six different models are used to simulate the propagation phase. Five of these models consider corrosion current as a

measure of deterioration rate. The corrosion rates predicted by these models are plotted in Figure 3.

Figure 3 shows that the corrosion rates predicted by various models are considerably different. To examine to what extent these differences affect structural performance predictions, different scenarios are considered. In all cases, parameters are the same and only the corrosion propagation model is different. To determine the serviceability limit state, probability density function of time to corrosion cracking and probability of corrosion cracking for different scenarios are estimated. The results are plotted in Figures 4 and 5.

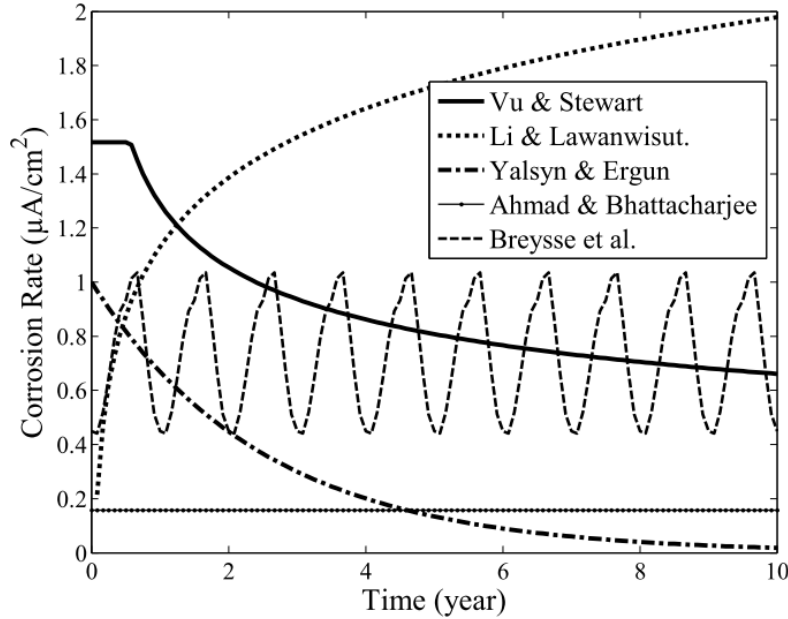


Fig. 3. Plots of different corrosion rate models.

Figure 4 illustrates no significant difference between the mean quantities of different time periods to corrosion cracking; the only exception is Ahmad and Bhattacharjee’s model. Figure 5 confirms that the probability of corrosion cracking is slightly different for the first four models in their early ages (<5 year), but for longer periods, results tend to be the same.

The similarity of the first four scenarios can be interpreted that: the mean of corrosion initiation time is 28.14 years. The time to corrosion cracking is about 1-2 years after corrosion initiation. As figure 3 shows, the corrosion rate of these models are pretty similar between the first and the second years. Therefore the time to corrosion cracking for these models is the same, too. Corrosion rate predicted by Ahmad and Bhattacharjee’s model is considerably lower than other models between the first and second years. The mean of corrosion cracking is higher in this scenario.

To investigate the effect of different corrosion propagation models on ultimate limit state, six different scenarios are considered and compared to the case of ‘no

deterioration’. Results for probability of failure versus time for different scenarios are shown in Figure 6.

It is observed from Figure 6 that the corrosion propagation phase is one of the most important factors in performance predictions; different deterioration models produce different results. The corrosion rates estimated by Ahmad and Bhattacharjee’s model and Yalsyn and Ergun’s model are very low, so there is no difference between the case of ‘no deterioration’ and the use of these models. In these cases, failure probability only marginally changes. As can be seen, however, Li’s model yields dramatically different results. Hence, prediction of the effect of deterioration processes on bridge safety is significantly sensitive to corrosion propagation phase model. Therefore, it is difficult to plan maintenance activities and determine the critical time for repair interventions.

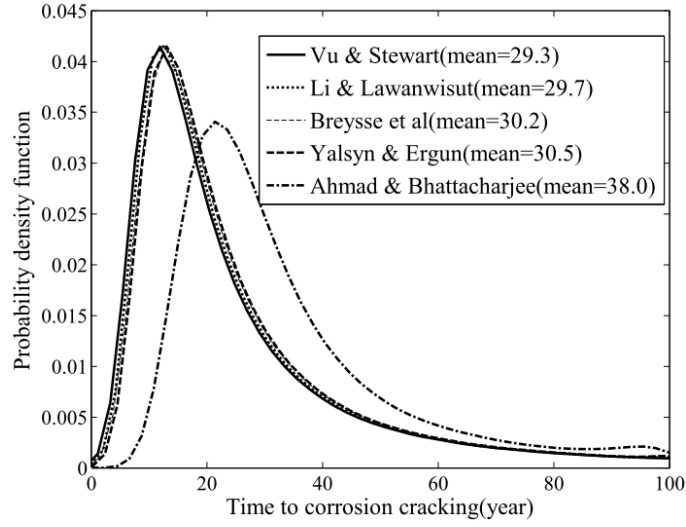


Fig. 4. Probability density function of time to corrosion cracking.

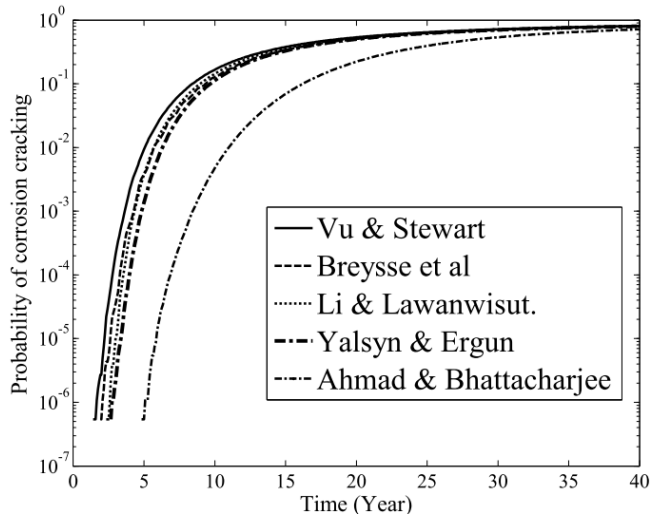


Fig. 5. Probability of corrosion cracking.

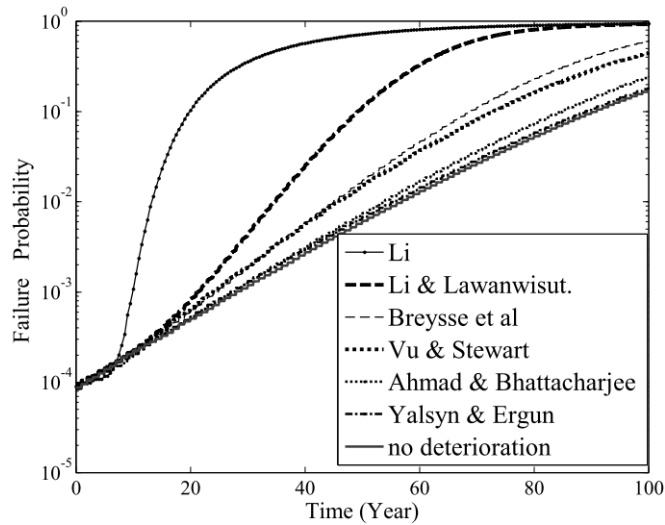


Fig. 6. Time-dependent failure probability.

DISCUSSION

The apparent drawback of these models is that they do not represent the actual corrosion conditions. These models are developed based on either electrochemical principles of corrosion or accelerated corrosion test results. It is crucial to validate these methods by using natural corrosion data. For instance Vu and Stewart's model does not consider temperature, which can affect the corrosion rate significantly. They developed a relationship between time and the corrosion rate based on experimental results reported by Liu and Weyers. While the data reported by Liu and Weyers is limited to corrosion processes where concrete resistivity (anodic reaction) is the governing reaction (Liu and Weyers, 1998), Vu and Stewart assumed that oxygen availability (cathodic reaction) is the governing reaction. Li and Lawanwisut's model also does not consider potential influencing parameters, including concrete characteristics and environmental conditions. In addition, the corrosion rate continuously increases with time, which does not correlate with the reported data in the existing literature (Liu and Weyers, 1998; Trejo and Monteiro, 2005). The model by Ahmad and Bhattacharjee considers several parameters that influence the corrosion rate, but such constant corrosion rate models are not representative of actual corrosion rates. Yalsyn and Ergun's model does not incorporate factors which affect corrosion rate such as cover depth, temperature, concrete resistivity and cracking. This model assumes that concrete and environmental conditions remain constant and corrosion rate is only dependent on time. However, these factors may vary from time to time and consequently affect corrosion rate. Breyssset al.'s model has the advantage of taking into account the effects of the climatic conditions

in the corrosion process. As the formation of rust products on the steel surface reduces the diffusion of the iron ions away from the steel surface, it is expected that the corrosion rate shrinks with time. However the result of Breyssset al.'s model changes with time frequently. Li's model generates unexpectedly high deterioration rates. It seems that this model is only applicable for the early ages after corrosion initiation and not for a long term prediction.

Based on the above findings, the magnitude of deterioration rate significantly alters performance prediction. Moreover, inaccurate modeling would result in an inaccurate appraisal of time-variant damages. Improving the accuracy of the corrosion propagation model improves the precision of the predicting models and consequently prediction of the service life of reinforced concrete structures would be more exact. Existing corrosion propagation models cannot reliably predict both the rate and severity of damages. In order to use these models as engineering tools for the design of new structures or to employ them to aid management decisions, further research is still required.

CONCLUSIONS

A structural deterioration reliability (probabilistic) model has been developed to calculate probabilities of structural failure. To this aim, six different models for propagation of corrosion have been considered. Results show that deterioration rates predicted by various models are significantly different. Considering prediction of the time to corrosion cracking, it is observed that different corrosion rate models produce slightly different results and this leads to considerably different results in ultimate limit state estimations. It is understood that the propagation phase is a critical phase for predicting corrosion in

reinforced concrete structures. As discussed, however, in respect to prediction activities, it is not treated as seriously as it deserves to be. The present challenges with the existing corrosion rate models prove there is need for the development of better models.

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