Whole life assessment of coastal RC structures

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ABSTRACT: A reliability-based method is presented in this paper for the whole life assessment of corrosion-affected reinforced concrete (RC) structures. Based on the experimental data produced from a comprehensive testing program, models are developed for corrosion-affected structural performance in various life cycles, i.e., corrosion initiation, corrosion-induced concrete cracking, excessive deflection of structural members and reduction of structural strength. A time-dependent reliability method is then employed to determine the probability of attaining each life cycle and each time period. The methodology presented in this paper can equip structural engineers and asset managers with confidence in decision-making with regard to the maintenance and repairs of RC structures.

1 INTRODUCTION

Reinforcement corrosion in concrete is regarded as a predominant causal factor in the premature deterioration of reinforced concrete (RC) structures, leading to ultimate structural failure (Broomfield 1997, Chaker 1992). Failure does not necessarily mean structural collapse only, but also includes loss of serviceability, characterized by concrete cracking, spalling, and excessive deflection. Clearly control and monitoring of reinforcement corrosion in concrete is of practical importance if premature failure of RC structures is to be prevented. Whole life performance assessment of RC structures is gradually becoming a requirement for the design of the new built RC structures and a necessity for decision-making with respect to inspection, repair, strengthening, replacement and demolition of aging and deteriorated RC structures (Enright & Frangopol 1998). There is therefore a clear need for determining the effects of the whole corrosion process on structural performance so that the service life of corroded RC structures can be predicted.

To meet the demand, various theoretical frameworks have been established to assess the whole life performance of corrosion-affected RC structures, including the employment of advanced reliability theories (Engelund et al. 1999, Frangopol et al. 1997, Maage et al. 1996). However, lack of rational models for structural deterioration in different cycles of the whole service life hampers the application of these advanced theories and in turn hinders the further progress of the framework. As a result, prediction of the service life of a structure remains at a stage of parametric studies. Prediction of the effects of the corrosion process, both initiation and propagation, on structural behaviour should be based on models derived from realistic and accurate data representative of service conditions (Enright & Frangopol 1998, Maage et al. 1996). Field data tends to be highly variable. Laboratory data is rarely produced from tests either with service loads or under natural salt ingress and simultaneous service loads or on full size structural members. This casts doubts on the applicability of the test results to concrete structures in service (Frangopol et al. 1997, Li 2000). In order to remedy this situation a comprehensive testing program has been undertaken (Li 2001, 2003) to produce data that closely represents the real service conditions of RC structures. From this data and complemented by data available in the research literature, rational and practical models of structural deterioration can be developed.

The intention of this paper is to present a reliability-based method for the whole life assessment of corrosion-affected RC structures. Based on the experimental data, models are developed for corrosion-affected structural performance. A time-dependent reliability method is employed to determine the probability of attaining each life cycle and each time period. The methodology presented in this paper can equip structural engineers and asset managers with confidence in making decisions with regard to the maintenance and repairs of RC structures.

2 FORMULATION LIFE CYCLES

For a corrosion-affected RC structure, a life cycle is defined a time period of the whole service life at the end of which actions of repairs and/or rehabilitation are required. As shown in Figure 1, five distinct life cycles can be identified as follows: (i) (0, T_i) from completion of a new built structure to corrosion initiation in the structure, (ii) (T_i, T_c) from corrosion initiation to corrosion-induced concrete cracking in the structure, (iii) (T_c, T_{cr}) from corrosion induced cracking to limit crack width in the structure, (iv) (T_{cr}, T_s) from limit crack width to loss of serviceability of the structure and (v) (T_s, T_f) from loss of serviceability to ultimate failure of the structure. In this paper failure is represented by loss of flexural strength of RC members.

With these definitions, the time period of each life cycle can be determined when models of structural performance in various cycles, i.e., corrosion initiation, concrete cracking, excessive deflection and structural failure, are available. In view of the complexity of the process of corrosion-induced structural deterioration and the lack of complete understanding of its effect on structural behaviour, it is very difficult to propose satisfactory theoretical models to assess RC structures. Instead, this paper resorts to developing empirical models of structural performance based on test results, which can be justified when the historical development of theories for RC structures is examined (Mirza et al. 1979).

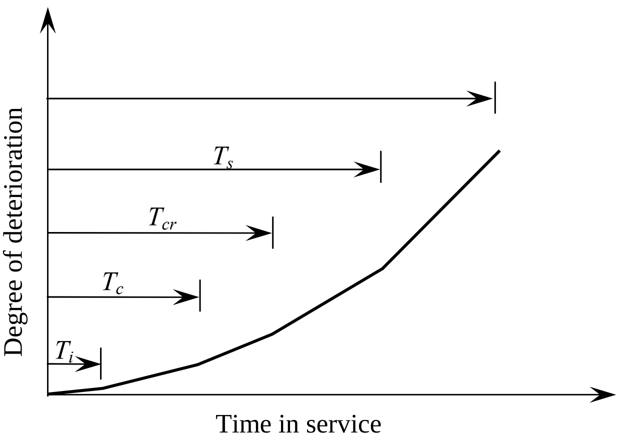

Figure 1. Life cycles of corrosion-affected structures.

3 CORROSION INITIATION

It is well known that chloride ingress in concrete is one of two basic mechanisms that trigger the corrosion of reinforcing steel in concrete. Since the corrosion onset is a random phenomenon, it is more reasonable to predict the corrosion initiation in a probabilistic manner

$$p_i(t) = P[C_{Cl}(t) \geq \delta_{Cl}] \times P[C_{Cl}] \tag{1}$$

where P is the probability of an event, $p_i(t)$ is the probability of corrosion initiation at time t, $P[C_{Cl}]$ is the probability of corrosion onset when the chloride content $C_{Cl}(t)$ is greater than the corresponding threshold value δ_{Cl}. Thus, for a given acceptable probability, $p_{i,a}$, the initiation time of reinforcement corrosion in concrete, T_i, can be determined by the following equation

$$p_i(T_i) \geq p_{i,a} \tag{2}$$

3.1 *Chloride ingress*

Due to the uncertainty of chloride ingress in concrete and its time-variant nature, it is well justified to model chloride ingress in concrete as a stochastic process. Based on the experimental results (Li 2001), the mean function $\mu_C(t)$ and coefficient of variation $V_C(t)$ of chloride content can be assumed to be in the form of

$$\mu_C(t) = C_0 \exp(at) \tag{3a}$$

$$V_C(t) = b \cdot t + 0.1433 \tag{3b}$$

where C_0 is the mean value of initial chloride content at the surface of steel bars and was found to be equal to 0.018 % of concrete weight; a and b are two coefficients and can be determined from experimental data from 183 specimens with a water-to-cement ratio of 0.45 that a = 0.01 and b = 0.0004 (Li 2001).

3.2 *Corrosion onset*

In the real world of RC structures, the onset of reinforcement corrosion is a random phenomenon and should be dealt with in a probabilistic manner as well. Based on the experimental investigations (Li 2001), the range of chloride content from 0.04 to 0.07 is the most sensitive to corrosion onset, which is consistent with the threshold values widely used in practice. From regression analysis of test results $P[C_{Cl}]$ can be expressed as (Li 2001)

$$P[C_{Cl}] = 9166C_{Cl}^3 - 321.4C_{Cl}^2 + 6.940C_{Cl} - 0.0013$$

$$\text{for } 0 \leq C_{Cl} \leq 0.03 \tag{4a}$$

$$P[C_{Cl}] = 8333C_{Cl}^3 - 2082C_{Cl}^2 + 172.3C_{Cl} - 3.731$$

$$\text{for } 0.03 < C_{Cl} \leq 0.1 \tag{4b}$$

4 CORROSION-INDUCED CRACKING

In predicting the time to surface cracking, the probability of corrosion-induced concrete cracking, $p_c(t)$, at time t is defined as

$$p_c(t) = P[\sigma_\theta(t) \geq f_t] \quad (5)$$

where $\sigma_\theta(t)$ is the tangential stress due to expansive corrosion products and f_t is the tensile strength of concrete. For a given acceptable probability, $p_{c,a}$, the time to surface cracking, T_c, can be determined by

$$p_c(T_c) \geq p_{c,a} \quad (6)$$

4.1 *Model of corrosion-induced cracking*

As is well known, concrete with embedded reinforcing bars can be modeled as a thick-wall cylinder (Bažant 1979) as shown in Figure 2(a), where D is the diameter of reinforcement bar, d_0 is thickness of the annular layer of corrosion products and C is the cover thickness. It is obvious that the inner radius x and the outer radius y of the thick-walled concrete cylinder are equal to $(D+2d_0)/2$ and $C+(D+2d_0)/2$, respectively. When the reinforcing steel corrodes in concrete, its products fill the pore band completely. As the corrosion progresses in concrete, a ring of corrosion products forms (Fig. 2(b)), the thickness of which, $d_s(t)$, can be determined as follows (Liu & Weyers 1998)

$$d_s(t) = \frac{W_{rust}}{\pi(D+2d_0)}\left(\frac{1}{\rho_{rust}} - \frac{\alpha}{\rho_{st}}\right) \quad (7)$$

where α is a coefficient related to types of rust products, ρ_{rust} is the density of corrosion products, ρ_{st} is the density of the steel. W_{rust} is the mass of corrosion products and can be expressed as (Liu and Weyers 1998)

$$W_{rust}(t) = \left(2\int_0^t 0.105(1/\alpha_{rust})\pi D i_{corr}(t)dt\right)^{1/2} \quad (8)$$

where $i_{corr}(t)$ is the corrosion current density (in $\mu A/cm^2$) which is a measure of corrosion rate. From the test results (Li 2003), $i_{corr}(t)$ can be expressed in terms of time t as

$$i_{corr} = -0.0016t^4 + 0.0333t^3 - 0.2483t^2$$
$$+0.8694t + 0.27 \quad (9)$$

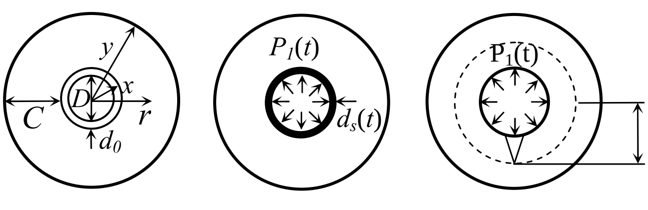

Figure 2. Schematic representation of cracking process.

When the tangential stress in concrete exceeds the tensile strength of concrete, cracks occur. After initial cracking, the crack in the concrete cylinder propagates along the radial direction and stops arbitrarily at r_0 (which varies between x and y) to reach a state of self-equilibrium (Fig. 2(c)). The crack divides the thick-wall cylinder into two co-axial cylinders: inner cracked and outer uncracked, as shown in Figure 2(c). Thus, the tangential stress at r_0 can be obtained that

$$\sigma_\theta(t) = \frac{P_1(t)x(t)}{r_0}\left[\frac{y^2 + r_0^2}{y^2 - r_0^2}\right] \quad (10)$$

From Equation (10), it is easily proven that, when r_0=0.486y, $P_1(t)$ reaches its maximum value. At this time, the crack width $w_1(t)$ at the inner surface of the concrete cylinder can be written as

$$w_1(t) = 2\pi x(t) - \pi D \quad (11)$$

5 CORROSION-INDUCED LIMIT CRACK WIDTH

Similarly, the probability of corrosion-induced concrete crack width, $p_{cr}(t)$, at time t is defined as

$$p_{cr}(t) = P[w(t) \geq w_{cr}] \quad (12)$$

where $w(t)$ is the crack width at the surface of the concrete cylinder and w_{cr} is the limit crack width prescribed in the building code. For a given $p_{cr,a}$, the time to limit crack width, T_{cr}, can be determined by

$$p_{cr}(T_{cr}) \geq p_{cr,a} \quad (13)$$

5.1 *Crack width evolution*

For the uncracked cylinder, it is obvious that, at r_0, the tangential stress is f_t and the radial compressive stress is $P_1(t)x(t)/r_0$. Thus, the corresponding tangential strain is equal to

$$\varepsilon_\theta = \frac{1}{E_c}(\sigma_\theta - \upsilon_c\sigma_r) = \frac{1}{E_c}\left(f_t + \frac{\upsilon_c P_1(t)x(t)}{r_0}\right) \quad (14)$$

where E_c is the elastic modulus of concrete and υ_c is the Poisson's ratio of concrete. The crack width at r_0 is then given by

$$w_2(t) = 2\pi r_0\varepsilon_\theta = \frac{2\pi r_0}{E_c}\left(f_t + \frac{\upsilon_c P_1(t)x(t)}{r_0}\right) \quad (15)$$

When concrete is considered to be a brittle material, the crack will penetrates through the cover once $P_1(t)$ reaches its maximum value. From Equations (11) and (15), the crack with at the concrete surface is given by

$$w(t) = w_1(t) + \frac{w_2(t) - w_1(t)}{r_0 - x(t)}[y - x(t)] \quad (16)$$

6 STRUCTURAL DETERIORATION

The reinforcement corrosion in concrete gradually reduces the capacity of RC structures and eventually leads to structural failure. The probabilities of loss of serviceability (i.e., excessive deflection of structural members), $p_s(t)$, and structural failure, $p_f(t)$, can be expressed as

$$p_s(t) = P[R(t) \le R_{a,s}] \quad (17a)$$

$$p_f(t) = P[R(t) \le R_{a,f}] \quad (17b)$$

where $R(t)$ is the structural resistance varying with time t, and $R_{a,s}$ and $R_{a,f}$ are the minimum acceptable serviceability and capacity, respectively. As proposed in Li (1995), a general model of structural deterioration can be of the form

$$R(t) = \varphi(t) R_0 \quad (18)$$

where $\varphi(t)$ is the deterioration function and R_0 is the original structural resistance. One of the advantages of the deterioration model in the form of Equation (18) is that the deterioration function φ is a relative rather than absolute value, i.e.,

$$\varphi(t) = \frac{R(t)}{R_o} \le 100\% \quad (19)$$

The relative form of the deterioration function can normalize the data from structures with different designed service and strength. This can maximize the use of available data that are usually scarce. With the introduction of the deterioration function, Equations (17a) and (17b) become

$$p_s(t) = P[\varphi_s(t) \le \varphi_{a,s}] \quad (20a)$$

$$p_f(t) = P[\varphi_u(t) \le \varphi_{a,f}] \quad (20b)$$

where $\varphi_s(t)$ and $\varphi_u(t)$ are the stiffness and strength deterioration functions, respectively, and $\varphi_{a,s}$ and $\varphi_{a,u}$ are the acceptable limits for stiffness and strength deterioration, respectively. For stiffness deterioration, the time to loss of serviceability, T_s, can be determined for an acceptable probability, $p_{s,a}$, by

$$p_s(T_s) \ge p_{s,a} \quad (21)$$

Similarly, the time to structural failure, T_f, can be determined for an acceptable probability of structural failure, $p_{f,a}$, by

$$p_f(T_f) \ge p_{f,a} \quad (22)$$

It should be noted that, besides safety/risk analysis for RC structures, determination of $p_{s,a}$ and $p_{f,a}$ involves social-economical considerations, which is beyond the scope of this paper.

6.1 *Crack width evolution*

Due to the randomness of structural deterioration and its time-variant nature, it is justifiable to model structural deterioration as a stochastic process, quantified by a deterioration function. Based on the tested results for RC flexural members located in a marine environment (Li 2001), the deterioration function can be modelled by a mean function, $\mu_\varphi(t)$, and a function of coefficient of variation, $V_\varphi(t)$, which can be assumed to be in the form of

$$\mu_\varphi(t) = \varphi_0 \exp(-\gamma t) \quad (23a)$$

$$V_\varphi(t) = \delta \cdot t + V_0 \quad (23b)$$

where φ_0 is initial (i.e., t = 0) deterioration function, which equals to unit, and γ is a coefficient representing the rate of structural deterioration. It allows for the effects of such factors as corrosion propagation, concrete composition and structural detailing. It may be appreciated that γ is difficult to determine in general. In Equation (23b), V_0 is the initial variation of concrete structural properties, which can be obtained from the research literature (Mirza et al. 1979), and δ is a coefficient representing the increase of uncertainty during the deterioration process. To embody the deterioration model of Equations (23a) and (23b) in practical use, mathematical regression is employed to process the experimental data (Li 2003). For stiffness deterioration factor, it is found that γ=0.096 and δ=0.014. It may be noted that V_0 equals 0.1 in Equation (23b) for stiffness deterioration. This is because that it is less certain that there is no stiffness deterioration at the beginning of the service of even new built structures, such as creep and shrinkage effects.

6.2 *Strength deterioration*

For RC flexural members located in a marine environment, the strength deterioration function can be modelled by a mean function, $\mu_\varphi(t)$, and a function of coefficient of variation, $V_\varphi(t)$, as shown in Equations (23a) and (23b). In this paper, based on regression analysis of the test results in Li (2003) which yields γ=0.0274 and δ=0.0161. V_0 in Equation (23b) can be assumed to be zero for strength deterioration because it is almost certain that there is no deterioration at the beginning of the service of new built structures.

7 CONCLUSION

A reliability-based method is presented in this paper for the whole life assessment of corrosion-affected reinforced concrete structures. Based on the experimental data collected from the literature, models are developed for corrosion-affected structural performance A time-dependent reliability method is employed to determine the probability of attaining each life cycle and each time period. The methodology presented in this paper can equip structural engineers and asset managers with confidence in making decisions with regard to the maintenance and repairs of RC structures.

ACKNOWLEDGEMENT

The financial supports from Engineering and Physical Sciences Research Council (EPSRC) of UK (No. GR/R28348) and from Hangzhou Bureau of Science and Technology of China (No. 200433134) are greatly acknowledged.

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