腐食鉄筋コンクリート部材の性能評価

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Performance Assessment of Corroded Reinforced Concrete Members

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1. INTRODUCTION

Reinforcement corrosion in concrete is the predominant causal factor for the premature deterioration of reinforced concrete (RC) structures, leading to structural failure (Broomfield 1997). According to the well-known Tuutti's model (1982) as shown in **Fig. 1**, the corrosion process in concrete structures includes two phases: initiation and propagation. Recent research has provided a wealth of evidence that for RC structures with load-induced cracks and subjected to constant chloride invasion, as represented by a marine environment, the initiation of reinforcement corrosion at the root of cracks occurs within a short period of time in service and the initiation time period is negligibly short if service life is considered (e.g., Francois and Castel 2001; Li 2003a).

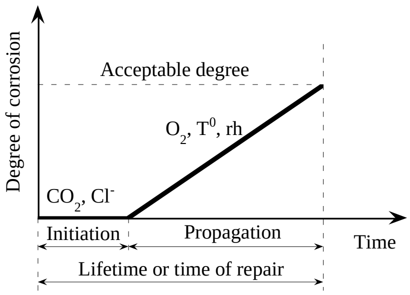

Fig. 1–Tuutti's service life model.

On the contrary, little research has been done on the corrosion propagation in concrete as measured by the corrosion rate or corrosion current density. In general, because the corrosion rate or corrosion current density varies in different structures, it should be obtained from site-specific measurement of the structure to be assessed over as long a period of time as practical. Obviously, the above method for determining the corrosion rate or corrosion current density is unrealistic. Therefore, Ting (1989) and Andrade et al. (1993) adopted a constant. Mori and Ellingwood (1994) and Frangopol et al. (1997) proposed a time-invariant random variable to represent it. Recently, Li (2003b; 2004) proposed a time-variant random variable for modeling the corrosion current density. However, a careful examination of the mean value of the time-variant random variable revealed that it is seriously underestimated. This is the reason that the corrosion current method grossly underestimates the strength deterioration of RC flexural members compared with those obtained from destructive load testing (Li 2003b). It is in this regard that the present paper attempts to develop a rational stochastic model of the corrosion rate and focuses on the structural strength deterioration as reinforcement corrosion propagates.

The intention of this paper is to present a reliability-based methodology for performance assessment of corrosion-affected RC structures as exemplified by RC flexural members subjected to chloride-induced reinforcement corrosion. A two-phase model of service life is developed with the focus on the deterioration of structural strength due to corrosion propagation. A stochastic model of the corrosion rate is proposed for representing corrosion propagation in RC structures. A comparison of the strength deterioration as determined by a destructive load test (Li 2001; Li 2003b) and the present corrosion rate method is undertaken, and it indicates that the corrosion rate method can effectively predict the strength deterioration of corrosion-affected RC structures. The time at which a corrosion-affected RC structure becomes unsafe,

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and hence requires strengthening or repair, can be predicted with the aid of the time-dependent reliability method. The methodology presented in this paper can be used as a tool by structural engineers and asset managers in assessing a corrosion-affected concrete infrastructure and making decisions with regard to its maintenance and rehabilitation. Timely maintenance and repairs have the potential to prolong the service life of corrosion-affected RC structures.

2. RESEARCH SIGNIFICANCE

The strength deterioration of corrosion-affected RC structures can be determined through two different methods: destructive load test and nondestructive measurement of corrosion rate. In practice, however, the strength deterioration can only be estimated based on the reduction of the cross-sectional area of reinforcing bars, and the reduction of the cross-sectional area of reinforcing bars is mainly depended on the corrosion rate. Therefore, in order to effectively predict the strength deterioration of corrosion-affected RC structures, it is necessary to develop a rational model of the corrosion rate. With the strength deterioration, methods of time-dependent reliability are employed to quantify the probability of failure so that the time the structure becomes unsafe, and hence requires repair, can be determined with confidence.

3. FORMULATION OF SERVICE LIFE

In this study, a service life comprising two phases is defined based on the deterioration of structural strength due to the reinforcement corrosion in RC structures. As schematically shown in **Fig. 2**, the first phase of the service life of a corrosion-affected RC structure is the time period from the completion of a newly built structure to the initiation of corrosion in the structure, denoted as $(0, T_i]$. The strength of the structure is seen as a constant during this period, and T_i can be predicted on the basis of recent research, such as that of Francois and Castel (2001) and Li (2003a). The second phase of the service life is the period from the initiation of corrosion to the final collapse of the structure (loss of strength), denoted as $(T_i, T_f]$. T_f can be determined once a performance-based assessment criterion is established. In the theory of structural reliability, the criterion can be expressed in the form of a limit state function as follows:

$$G(R,S,t) = R(t) - S(t) \tag{1}$$

where $R(t)$ denotes the structural resistance at time t, which can be modeled as a time-variant random variable or a stochastic process; $S(t)$ denotes the load effect at time t, which is generally can be modeled as a stochastic process and is usually assumed to be a time-invariant random variable; and $G(R, S, t)$ is the performance function.

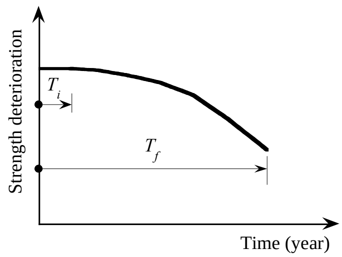


Fig. 2–Schematic model of life cycles of corrosion-affected RC structures.

Commonly, with the performance function of Eq. 1, the probability of failure, $P_f(t)$, can be determined by

$$P_f(t) = P[G(R,S,t) \le 0] = P[R(t) - S(t) \le 0] \tag{2}$$

Eq. 2 represents a typical upcrossing problem, which can be dealt with by time-dependent reliability methods.

When $P_f(t)$ is greater than the maximum acceptable risk in terms of the probability of failure, i.e., P_a, or the reliability index $\beta(t)$ corresponding to $P_f(t)$ is less than the reliability index β_a corresponding to P_a, then the structure becomes unsafe and requires repairs. This can be determined from the following equation:

$$P_f(T_f) \ge P_a \quad \text{or} \quad \beta(T_f) \le \beta_a \tag{3}$$

where T_f denotes the time the structure becomes unsafe.

4. A STOCHASTIC MODEL OF THE CORROSION RATE

Typical corrosion rates of steel in various environments have been reported in recent years. According to Ting (1989), the average corrosion rate v_{corr} for passive steel in concrete attacked by chloride is about 100 m/year (3.94×10^{-3} in./year). From Mori and Ellingwood's research (1994), the typical corrosion rate v_{corr} is a time-invariant random variable described by a lognormal distribution with a mean of 100 m/year (3.94×10^{-3} in./year) and a coefficient of variation of 0.5. A time-invariant random variable with a mean of 89 m/year (3.51×10^{-3} in./year) and a standard deviation of 25 m/year (9.85×10^{-4} in./year) for modeling v_{corr} was suggested by Frangopol et al. (1997). Based on the experimental data obtained under a simulated marine condition for flexural members (Li, 2001), the corrosion current density I_{corr} is assumed to be a time-invariant random variable with the coefficient of variation of 0.2 by Li (2004), which is expressed as

$$I_{corr} = 0.3683\ln(t) + 1.1305 \tag{4}$$

where t = time in years.

If t = 4, 8, and 12 years, the values of corrosion current density can be readily obtained as 1.6411, 1.7904, and 1.8964 A/cm^2 (1.5246, 1.6633, and 1.7618 mA/ft^2). Using the conversion equation 1 A/cm^2 (0.929 mA/ft^2) = 11.6 m/year (4.57×10^{-4} in./year) (Li 2003b), the values of corrosion rate can be readily obtained as 19.0364, 20.7687, and 21.9978 m/year (7.5×10^{-4}, 8.18×10^{-4}, and 8.67×10^{-4} in./year), which are far from the values adopted by other researchers above.

As may be appreciated, I_{corr} can only be obtained from site-specific measurement of the structure to be assessed. In this paper, considering the complexity and randomness of corrosion propagation in corroded RC structures, a stochastic model of the corrosion rate is proposed as

$$v_{corr}(t) = L_1\ln(t) + L_2 \tag{5}$$

where L_1 and L_2 are independent time-invariant lognormal random variables with mean values of μ_{L1} = 2.0×10^{-3} cm/year (7.87×10^{-4} in./year) and μ_{L2} = 6.0×10^{-3} cm/year (2.364×10^{-3} in./year) and coefficients of variation of $V_{L1} = V_{L2} = 0.2$.

The mean and standard deviation functions of $v_{corr}(t)$ are

$$\mu_{v_{corr}}(t) = \mu_{L_1}\ln(t) + \mu_{L_2} \tag{6}$$

$$\sigma_{v_{corr}}(t) = \sqrt{\sigma_{L_1}^2\ln^2(t) + \sigma_{L_2}^2} \tag{7}$$

The mean and coefficient of variation of the corrosion rate $v_{corr}(t)$ are illustrated in **Fig. 3**. As can be seen, there is a sharp increase of the mean corrosion rate in the first time period and in the mean time, the coefficient of variation decreases quickly, which can effectively reflect the test results obtained by Li (2001, 2003b). As time goes on, both the mean and the coefficient of variation of corrosion rate tend to be moderate.

5. DETERIORATION OF STRUCTURAL STRENGTH

The flexural strength of corrosion-affected concrete members depends mainly on the total available area of longitudinal rebars in the tension zone. The uniform corrosion of a reinforcing bar in RC concrete structures is shown in **Fig. 4**. From the figure, the total bending reinforcement area as a function of time t, $A_L(t)$, can be expressed as

$$A_L(t) = \begin{cases} \dfrac{n\pi D_L^2}{4} & \text{for } t \le T_i \\ \dfrac{n\pi[D_L - 2\int_{T_i}^{t} v_{corr}(t)\,dt]^2}{4} & \text{for } t > T_i \end{cases} \tag{8}$$

where D_L (cm) (1cm = 0.394 in.) = diameter of a longitudinal rebar; n = number of bars; T_i = time of corrosion initiation; and $v_{corr}(t)$ = rate of corrosion.

Substituting Eq. 5 in Eq. 8 produces

$$A_L(t) = \begin{cases} \dfrac{n\pi D_L^2}{4} & \text{for } t \le T_i \\ \dfrac{n\pi\{D_L - 2[L_1 t\ln t + (L_2 - L_1)t - L_1 T_i \ln T_i - (L_2 - L_1)T_i]\}^2}{4} & \text{for } t > T_i \end{cases} \tag{9}$$

As has been described, the initiation time period is negligibly short if the service life is considered and as will be shown in the example that the different values of T_i has only a slight effect on the structural failure; T_i is suggested to be equal to 1.0 year in this paper. Then Eq. 9 can be simplified as:

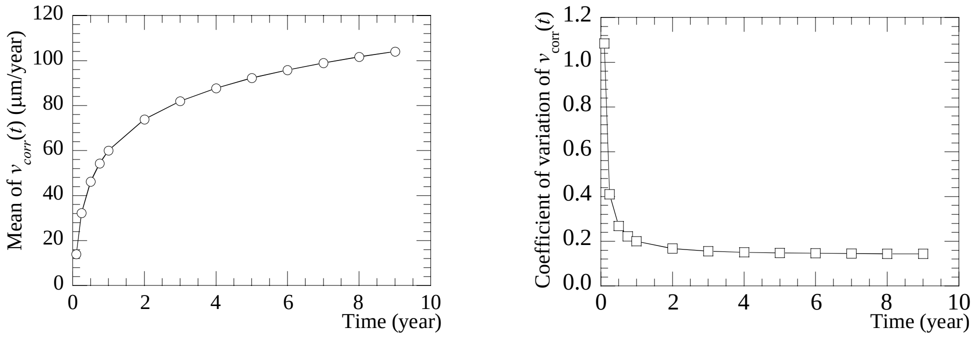
$$A_L(t) = \begin{cases} \dfrac{n\pi D_L^2}{4} & \text{for } t \le 1.0 \\ \dfrac{n\pi\{D_L - 2[L_1 t\ln t + (L_2 - L_1)(t-1)]\}^2}{4} & \text{for } t > 1.0 \end{cases} \tag{10}$$

The flexural strength of a RC beam at time t, $M_R(t)$, may be approximately computed as (AIJ 1991)

$$M_R(t) = 0.9A_L(t)F_{yL}H_0 \tag{11}$$

where F_{yL} = the yield strength of a longitudinal reinforcing bar;

and H_0 = the effective depth.



(a) (b)

Fig. 3–Corrosion propagation as measured in corrosion rate $v_{corr}(t)$ (1.0 m/year = 3.94×10^{-5} in./year).

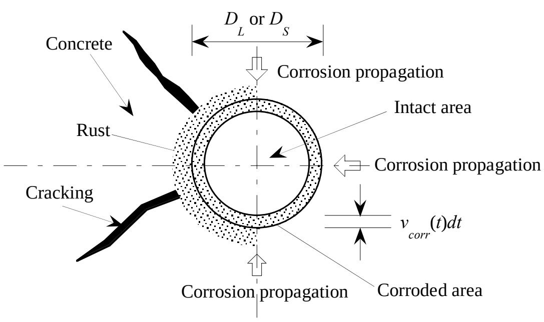


Fig. 4–Uniform corrosion of a reinforcing bar in RC concrete members.

In this analysis, the ratio of the mean damaged capacity (deterioration strength), $\mu_{MR(t)}$, to the mean intact capacity (original structural strength), $\mu_{MR(0)}$, is defined as the residual capacity function (deterioration function)

$$C_R(t) = \frac{\mu_{M_R(t)}}{\mu_{M_R(0)}} \tag{12}$$

According to Eq. 11, the mean flexural strength of the corrosion-affected RC members at a time t can be calculated using the Monte-Carlo simulation (Melchers 1999). A comparison of strength deterioration as determined from destructive load testing (Li 2001; Li 2003b) and that obtained using the present corrosion rate method is shown in **Fig. 5**. As can be seen, the results obtained by the present corrosion method are in close agreement with those obtained from the destructive load experiments. This indicates that the present corrosion rate method can effectively predict the strength deterioration of RC flexural members.

On the other hand, if only stirrups for shear reinforcement are used, the shear strength depends on the reinforcement placed perpendicular to the axis of the member. Under uniform corrosion (also see **Fig. 4**), the cross-sectional area of a stirrup as a function of time, $A_S(t)$ is given by

$$A_S(t) = \begin{cases} \dfrac{2\pi D_S^2}{4} & \text{for} \quad t \le T_i \\ \dfrac{2\pi[D_S - 2\int_{T_i}^{t} v_{corr}(t)dt]^2}{4} & \text{for} \quad t > T_i \end{cases} \tag{13}$$

where DS (cm) (1cm = 0.394 in.) = diameter of a stirrup.

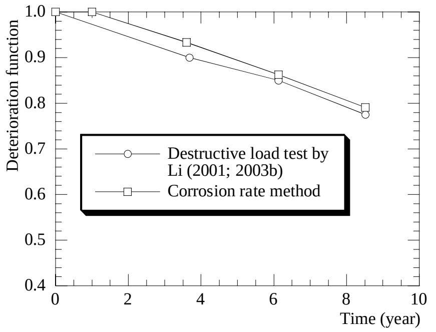


Fig. 5–Comparison of strength deterioration determined by different methods.

Again, substituting Eq. 5 in Eq. 13 produces

$$A_S(t)=\begin{cases}\dfrac{2\pi D_S^2}{4} & t\le T_i\\ \dfrac{2\pi\{D_S-2[L_1 t\ln t+(L_2-L_1)t-L_1T_i\ln T_i-(L_2-L_1)T_i]\}^2}{4} & t>T_i\end{cases} \quad (14)$$

Similarly, when T_i =1.0 year,

$$A_S(t)=\begin{cases}\dfrac{2\pi D_S^2}{4} & \text{for}\quad t\le 1.0\\ \dfrac{2\pi\{D_S-2[L_1 t\ln t+(L_2-L_1)(t-1)]\}^2}{4} & \text{for}\quad t>1.0\end{cases} \quad (15)$$

Thus, the time-dependent shear strength is

$$V(t)=V_C+V_S(t) \quad (16)$$

In Eq. 16, it is assumed that the shear strength of concrete V_C is time-independent. According to AIJ (1991), V_C can be computed by using the following equation.

$$V_C=\frac{7}{8}H_0B\alpha F_s \quad (17)$$

where B = the width of a beam; α = an adjustment factor depending on the ratio of shear span to depth; and F_s = the allowable shear stress of concrete.

When only shear reinforce perpendicular to the axis of the member is used, the shear strength due to stirrups is given as

$$V_S(t)=\frac{A_S(t)F_{yS}H_0}{S} \quad (18)$$

where F_{yS} = the yield strength of a stirrups; and S = spacing between stirrups.

6. RELIABILITY ANALYSIS OF CORROSION-AFFECTED RC BEAMS

Consider a one-story one-bay RC frame located in a marine environment as shown in **Fig. 6**. The performance function corresponding to the flexural strength failure mode can be expressed as

$$G_1(\mathbf{X})=0.9A_L(t)F_{yL}H_0-\frac{(W_D+W_L+W_S)}{8}L^2 \quad (19)$$

where $A_L(t)$ is a time-dependent random variable; W_D= the dead load; W_L= the live load; and W_S= the snow load. The probability characteristics of the basic random variables are listed in **Table 1**.

The time-dependent reliability indexes obtained by using FORM (Melchers 1999) with respect to the service time are depicted in **Fig. 7a**, together with two different times of corrosion initiation, i.e., T_i = 0.2, 0.6 years. As can be seen from the figure, the reliability index decreases as the reinforcement corrosion in the RC structure propagates. Also, one can see that the time of initial corrosion has only a slight effect on the results.

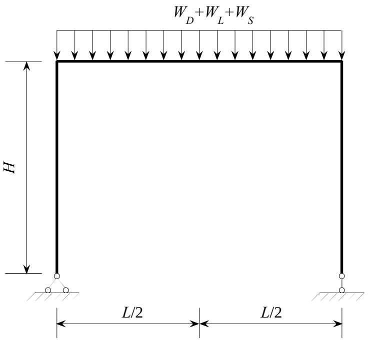
For the shear strength failure mode, the performance function is given as

$$G_2(\mathbf{X})=\frac{7}{8}H_0B\alpha F_s+\frac{A_S(t)F_{yS}H_0}{S}-\frac{(W_D+W_L+W_S)}{2}L \quad (20)$$

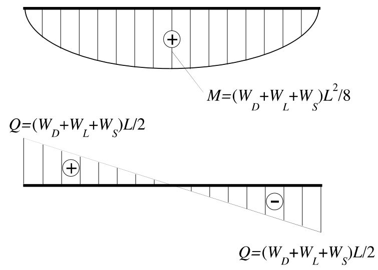
where $A_S(t)$ is a time-dependent random variable and the probability characteristics of the other basic random variables are also listed in **Table 1**. Here, the factor α is seen as a constant instead of a random variable. According to AIJ (1991), α is calculated as 1.0 in this example.

Similarly, the time-dependent reliability indexes corresponding to the shear failure mode obtained by using FORM with respect to the service time are depicted in **Fig. 7b**, together with two different times of corrosion initiation, i.e., T_i = 0.2, 0.6 years. From the figure, one can see that the reliability index decreases as the reinforcement corrosion in the RC structures propagates. The reliability index is relatively large for this example because the load effect is relatively small. Again, one can see that the time of corrosion initiation has only a slight effect on the results.

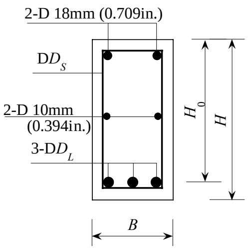
From **Fig. 7a** and **7b**, it is not difficult to understand that the beam is much more likely to collapse from the flexural failure mode.



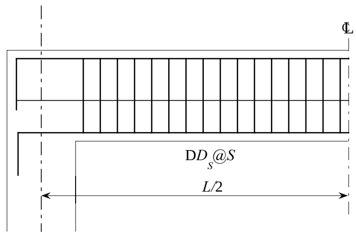
(a) A one-story one-bay frame structure



(b) The moment and shear diagrams

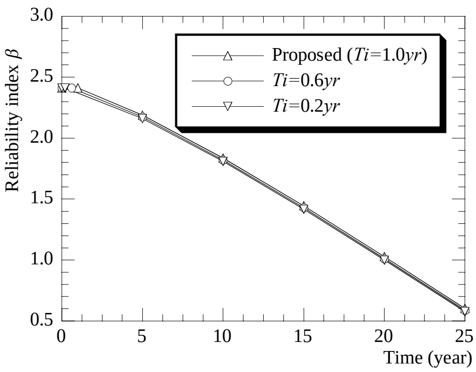


(c) Cross-section with maximum moment

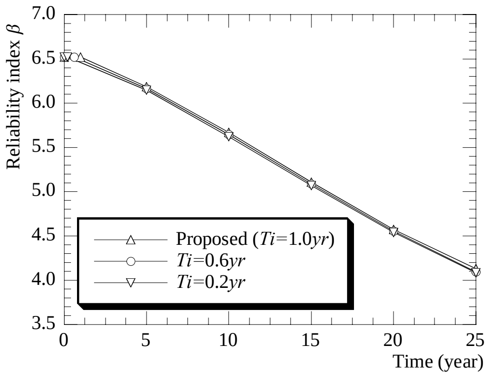


(d) Longitudinal section of beam

Fig. 6–A corrosion-affected reinforced concrete frame beam.



(a) flexural strength failure mode



(b) shear strength failure mode

Fig. 7–Time-dependent reliability indexes.

Table 1–Basic variables and their probability characteristic.

Basic variables	Mean	Standard deviation	Distributions
$v_{corr}(t) = L_1 \ln(t) + L_2$			
L_1	2.0×10^{-3} cm/year (7.87×10^{-4} in./year)	4.0×10^{-4} cm/year (1.576×10^{-4} in./year)	Lognormal
L_2	6.0×10^{-3} cm/year (2.364×10^{-3} in./year)	1.2×10^{-3} cm/year (4.728×10^{-4} in./year)	Lognormal
$\boldsymbol{B}$	25 cm (9.843 in.)	0.25cm (0.098 in.)	Normal
H_0	53 cm (20.866 in.)	0.5cm (0.197 in.)	Normal
L	720 cm (283.46 in.)	20 cm (7.87 in.)	Normal
$\boldsymbol{D_L}$	2.2 cm (0.866 in.)	0.1cm (0.0394 in.)	Normal
$\boldsymbol{D_S}$	1.0 cm (0.394 in.)	0.05cm (0.0197in.)	Normal
S	20 cm (7.87 in.)	2 cm (0.787 in.)	Normal
F_{yL}	350 MPa (5.07×10^4 psi)	35 MPa (5.07×10^3 psi)	Lognormal
F_{yS}	250 MPa (3.62×10^4 psi)	25 MPa (3.62×10^3 psi)	Lognormal
F_s	1.05 MPa (152.17 psi)	0.21MPa (15.22 psi)	Lognormal
$\boldsymbol{W_D}$	10 kN/m (0.685 kip/ft)	1 kN/m (0.069 kip/ft)	Normal
W_L	2 kN/m (0.137 kip/ft)	0.8 N/m (0.055 kip/ft)	Lognormal
$\boldsymbol{W_S}$	2.5 kN/m (0.171 kip/ft)	1.75 kN/m (0.12 kip/ft)	Gumbel

7. PREDICTION OF SERVICE LIFE OF CORROSION-AFFECTED RC BEAMS

With the proposed models of structural strength deterioration, it is not difficult to predict the time for corrosion-affected RC structures to be unsafe once the acceptance criteria is determined in Eq. 3. However, it is quite difficult to decide an acceptable limit for the strength deterioration since the determination of it involves social-economical considerations, in addition to risk analysis for the structure. Clearly, different acceptance criteria will result in different times for the structure to be unsafe. This is the risk involved in decision-making. Gonzales et al. (1996) observe that a damage level of 25% in terms of the reduction in cross-sectional area of reinforcement bars seems to be prominent in corrosion-affected RC structures. This observation is based on data from the Eurointernational Committee of Concrete which classifies structural deterioration according to external signs, such as rust spots, concrete cracks, and cover delamination, as well as reduction of the cross-sectional area of reinforcement bars. Amey et al. (1998) predict the service life of corrosion-affected concrete structures using a more simplistic 30% rebar reduction as the failure criterion. Li (2003b) takes the acceptable limit for strength deterioration as 0.6 as similarly defined in Eq. 12 and predicts the time at which the structure becomes unsafe using a confidence level of 90%. Because the collapse of the frame beam will result in a serious loss of property and life, the minimum acceptable risk in terms of the reliability index, β_a, is taken as 1.0, and the time for the structure to become unsafe (failure) T_f, can be predicted from Eq. 3, i.e.,

$$\beta(T_f) \le \beta_a = 1.0 \tag{21}$$

which gives T_f= 20.31 years. It is apparent that, compared with the time of structural failure, the initial time of corrosion (T_i= 1.0 year) is negligibly short, as reported by other researchers.

8. CONCLUSIONS

1) A stochastic model of the corrosion rate is proposed. A comparison of the strength deterioration, as determined from a destructive load test, and the present corrosion rate method reveals that the present corrosion rate method can effectively predict the strength deterioration of corrosion-affected RC structures.
2) A two-phase model of service life is developed. The initial time of corrosion, i.e., T_i= 1.0 year, is suggested. Methods of time-dependent reliability are employed to quantify the reliability index (probability of failure) so that at which the time of a corroded RC structure becomes unsafe and hence requires repair (T_f) can be determined with confidence.
3) The methodology presented in the paper can be used as a tool for structural engineers and asset managers to assess a corrosion-affected concrete infrastructure and make decisions with regard to its maintenance and rehabilitation.

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REFERENCES

1. AIJ, Standard for Structural Calculation of Reinforced Concrete Structures, Tokyo, Japan, 1991. (In Japanese)
2. Amey, S.L., Johnson, D.A., Miltenberger, M. A., and Farzam, H., “Predicting the service life of concrete marine structures: An environment methodology,” ACI Materials Journal, V.95, No.2, 1998, pp205-214.
3. Andrade, C., Alonso, C., and Molina, F.J., “Cover cracking as a function of bar corrosion: Part I–Experiment test,” Materials and structures, 26, pp453-464, 1993.
4. Broomfield, J., Corrosion of steel in concrete, understanding, investigating & repair, E & FN Spon, London, 1997.
5. Francois, R., and Castel, A., “Discussion on influences of bending crack and water-cement ratio on chloride-induced corrosion of main reinforcing bars and stirrups,” ACI Materials Journal, V.98, No.3, 2001, pp276-278.
6. Frangopol, D. M., Lin, K. Y., and Estes, A., “Reliability of reinforced concrete girders under corrosion attack,” Journal of Structural Engineering, ASCE, V.123, No. 3, 1997, pp286-297.
7. Gonzales, J. A., Feliu, S., Rodriguez, P., Lopez, W., Alonso, C., and Andrade, C., “Some questions on the corrosion of steel in concrete. II: Corrosion mechanism and monitoring, service life prediction and protection methods,” Materials Structure, V.29, 1996, pp97-104.
9. Li, C. Q., “Initiation of chloride induced reinforcement corrosion in concrete structural members-Experiment,” ACI Structural Journal, V.98, No.4, 2001, pp501-510.
10. Li, C. Q., “Life-cycle modeling of corrosion-affected concrete structures — Initiation,” Journal of Materials in Civil Engineering, ASCE, V.15, No. 6, 2003a, pp594-601.
11. Li, C. Q., “Life-cycle modeling of corrosion-affected concrete structures — Propagation,” Journal of Structural Engineering, ASCE, V.129, No.6, 2003b, pp753-761.
12. Li, C. Q., “Reliability based service life prediction of corrosion-affected concrete structures,” Journal of Structural Engineering, ASCE, V.130, No.10, 2004, pp1570-1577.
13. Melchers, R. E., Structural reliability analysis and prediction, Wiley, Chichester, UK, 1999.
14. Mori, Y., and Ellingwood, B. R., “Maintaining reliability of concrete structures II: optimum of inspection/repair,” Journal of Structural Engineering, ASCE, V.120, No.3, 1994, pp846-862.
15. Ting, S.-C., “The effects of corrosion on the reliability of concrete bridge girders.” PhD thesis, 1989, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.
16. Tuutti, K., Corrosion of steel in concrete, Swedish Cement and Concrete Research Institute, pp17-21, 1982.