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Risk-based optimum repair planning of corroded reinforced concrete structures

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Abstract. Civil engineering infrastructure is aging and requires cost-effective maintenance strategies to enable infrastructure systems operate reliably and sustainably. This paper presents an approach for determining risk-cost balanced repair strategy of corrosion damaged reinforced concrete structures with consideration of uncertainty in structural resistance deterioration. On the basis of analytical models of cover concrete cracking evolution and bond strength degradation due to reinforcement corrosion, the effect of reinforcement corrosion on residual load carrying capacity of corroded reinforced concrete structures is investigated. A stochastic deterioration model based on gamma process is adopted to evaluate the probability of failure of structural bearing capacity over the lifetime. Optimal repair planning and maintenance strategies during the service life are determined by balancing the cost for maintenance and the risk of structural failure. The method proposed in this study is then demonstrated by numerical investigations for a concrete structure subjected to reinforcement corrosion. The obtained results show that the proposed method can provide a risk cost optimised repair schedule during the service life of corroded concrete structures.

Keywords: reinforcement corrosion; residual strength; stochastic deterioration modelling; time-dependent reliability; optimised maintenance strategy

1. Introduction

Deterioration of reinforced concrete (RC) structures due to reinforcement corrosion is a growing problem worldwide. It typically involves cracking and spallling of concrete cover and reduction in area of steel reinforcement and loss of bond between concrete and corroded rebar. This eventually affects the service life of the concrete structures and also increases the resources required for the maintenance and rehabilitation over time. Thus, maintenance and management of corrosion affected RC structures has become a worldwide issue for engineers and asset managers (Chen and Alani 2013, Papakonstantinou and Shinozuka 2013, Tilly and Jacobs 2007). For existing corrosion affected RC structures, there is a need for systematic structural health monitoring strategy and risk-based condition assessment techniques, which can confidently predict how these deteriorating structures will respond to the deterioration and when they will fail to deliver the required capacity (Chen and Alani 2013, Nepal and Chen 2014a). The integration of structural performance assessment methods and time-dependent reliability analysis techniques has

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tremendous potential in providing cost-effective maintenance strategy of aging structures. Therefore, the need for reliability-based performance assessment integrating with the optimised maintenance strategy is evident for the sustainable infrastructure management.

A characteristic feature of optimising maintenance is that the balanced decisions need to be made under uncertainties in many factors such as structural performance deterioration. In maintenance management, the most critical uncertainty is generally the uncertainty in the time to structural failure (lifetime) and/or the rate of deterioration (Van Noortwijk and Frangopol 2004, Van Noortwijk 2009, Chen and Bicanic 2010). In order to incorporate this, a stochastic deterioration model based time-dependent reliability analysis is necessary and should be used to determine the optimal maintenance strategy (Liu and Frangopol 2005, Van Noortwijk 2009, Chen and Xiao 2014, Tee *et al*. 2013, Van Noortwijk and Frangopol 2004).

This paper presents an approach for optimising maintenance strategy by using time-dependant reliability analysis and lifecycle cost analysis techniques. In order to optimise maintenance strategy represented here as repair time, the maintenance model based on the risk cost balanced criteria is employed. Analytical solutions are provided to evaluate the flexural strength deterioration due to reinforcement corrosion. In order to model the progression of flexural strength deterioration during the life cycle of the RC structure, a gamma process model is adopted. The time-dependant reliability analysis is then applied to evaluate the probability of failure over life cycle. In the reliability analysis, structural failure is defined when its deterioration reaches the predefined allowable limit. Finally, the annual probability of failure evaluated from the time-dependant reliability analysis is employed to determine the optimised maintenance strategy. The applicability of the proposed approach is demonstrated by a numerical example.

2. Concrete cover under reinforcement corrosion

The progress of corrosion directly affects structural performance and hence the remaining service life of a corroding RC structure. The quantitative description of these damages associated with performance deterioration of corrosion damaged RC structures is the first step in reliability analysis of these structures. Research showed that the corrosion products formed during corrosion process are expansive in nature, which causes two to six times volume increase as compared with the original steel (Papakonstantinou and Shinozuka 2013). The increase of volume per unit length due to bar corrosion ΔV can be obtained from the volume of rust minus the volume of the original rebar of a diameter R_b consumed. This increment of volume per unit length of rebar creates a radial displacement at the rebar-concrete interface u_{bx} is expressed here as

$$u_{bx} = \frac{\Delta V}{2\pi R_b} = \frac{1}{2}\left(\gamma_{vol} - 1\right) R_b X_p \tag{1}$$

where X_p is the corrosion level defined by the ratio of the mass loss of the corroded rebar to the original mass of the rebar and γ_{vol} is the volume ratio of the corrosion product formed to its parent metal between 1.8 to 6.4. In this paper corrosion has been considered as uniform, therefore, reduction in rebar radius from the initial state R_b when uniform attack penetration x occurs can be evaluated from $R_{bx} = R_b - x$. The evolution of cracks in concrete cover is discussed in the analytical investigations by Chen and Alani (2013) and Chen and Xiao (2012), where the equivalent cracks width over the time was defined as the cumulated crack width over the cover surface. The intact cover concrete is treated as elastic material and the cracked concrete is considered as anisotropic in nature. From the anisotropic property and the bilinear softening law of the cracked concrete, normalised cumulative crack width over the concrete cover surface is obtained by considering boundary conditions and by ignoring the Poisson's effect associated with the hoop strain of the completely cracked concrete, expressed here as

$$W_{cx} = \frac{\frac{E}{f_t} u_{bx} - a\left[R_b + R_c\left(l_o - R_c\right)\left(l_o - R_b\right).\delta\left(R_c, R_b\right)\right]}{b\left(l_o - R_b\right)\left[1 - R_c\left(l_o - R_c\right).\delta\left(R_c, R_b\right)\right]} \tag{2}$$

where l_o is the material constant given by $l_o = n_c l_{ch} / 2\pi b$ in which n_c is the number of cracks taken as 3 or 4 for the concrete around the rebar and l_{ch} is the characteristic length defined as $l_{ch} = EG_f / f_t^2$ by Hillerborg *et al*. (1976); and $\delta\left(R_c, R_b\right)$ is the crack factor associated with the material properties and radial distance r between rebar surface R_b and concrete cover surface R_c, defined as.

$$\delta(R_c, R_b) = \frac{R_c - R_b}{l_o(l_o - R_c)(l_o - R_b)} + \frac{1}{l_o^2} \ln \frac{R_c}{R_b} \frac{|l_o - R_b|}{|l_o - R_c|} \tag{3}$$

3. Corrosion induced flexural strength deterioration

The flexural strength deterioration due to reinforcement corrosion was investigated by Nepal and Chen (2014b), where the analytical model was proposed to evaluate the residual strength of RC beam with corroded reinforcement by considering different failure modes of RC structures. In case of un-corroded perfectly bonded RC beam, the strain compatibility condition exists, as given by design codes. Therefore, the initial flexural resistance of RC beams can be evaluated by using design codes. For the corroded RC beam when ultimate bond strength is insufficient to prevent anchorage failure, the tensile force generated in the corroded tensile rebar can be obtained from

$$f_{stx} = 2 n_b \pi R_{bx} l_d T_{ubx} \tag{4}$$

where n_b is the number of the bottom tensile steel and l_d is the development length which can be evaluated from design code. T_{ubx} is the ultimate bond strength of corroded rebar and is given by Nepal and Chen (2014b). The strain compatibility of a RC beam with corroded reinforcement can be considered between un-bonded and bonded condition (Wang and Liu 2010). Assuming the deformation of concrete is mainly due to plastic deformation occurring within the plastic equivalent region, new strain compatibility of the corroded beam can be expressed as

$$\frac{\varepsilon_{stx}}{\varepsilon_{ccx}} = g_x \frac{d_x - Y_x}{Y_x}, \qquad \frac{\varepsilon_{scx}}{\varepsilon_{ccx}} = g_x \frac{Y_x - d'_x}{Y_x} \tag{5}$$

where the plastic equivalent region is defined as $L_{eq} = 9.3 Y_x$ (Au and Du 2004). Parameters in Eq. (5) are defined as: ε_{ccx} is ultimate strain of concrete; ε_{stx} and ε_{scx} are strains of tensile steel and compression steel, respectively; Y_x is the neutral axis depth from the edge of compression zone; d_x is the effective depth of beam and $d_x^{'}$ is the distance from the centroid of the compression steel to edge of the compressive fibre corresponding to corrosion level X_p; and g_x is the interpolation factor which can be obtained by considering the bond strength value of perfectly bonded and un-bonded condition of the RC beam. By utilising the concept given by Cairns and Zhao (1993) that the corroded RC beam still follows the condition of equilibrium of resultant tensile and compressive forces acting at the beam section, the residual flexural strength can be evaluated by

$$M_{ux} = f_{ccx}\left(d_x - 0.4Y_x\right) + f_{scx}\left(d_x - d_x^{'}\right) \tag{6}$$

where f_{ccx} and f_{scx} are the compressive forces acting at the centroid of compression zone and the centroid of the compressive steel of the corroded beam, respectively.

4. Stochastic deterioration modelling and probability of failure

The gamma process has been often adopted for structural deterioration modelling (Chen and Alani 2013, Van Noortwijk 2009, Van Noortwijk and Frangopol 2004). The gamma process is a stochastic process with independent non-negative increments having a gamma distribution with a given average of deterioration rate. Structural resistance degradation caused by reinforcement corrosion is a continuous and non-negative phenomenon. Therefore, the gamma process is suitable for the stochastic modelling of structural resistance deterioration in corrosion affected RC structures during their life cycle. In the gamma process deterioration model, cumulative resistance deterioration J is considered as a random quantity with the gamma distribution, and has the shape parameter $\eta_x > 0$ and scale parameter $\lambda > 0$. Then, the probability density function of this random quantity J, i.e. the structural resistance during the life cycle at time t and corrosion level X_p $(X_p > 0)$, can be formulated as

$$f_{J_x}(J) = Ga(J, \eta_x, \lambda) = \begin{cases} \dfrac{\lambda^{\eta_x}}{\Gamma \eta_x} J^{\eta_x - 1} e^{-\lambda J}, & for\, J \geq 0 \\ 0, & elsewhere \end{cases} \tag{7}$$

where $\Gamma(\eta_x) = \int_0^{\infty} v^{\eta_x - 1} e^{-v} dv$ is the gamma function for shape parameter $\eta_x > 0$. The scale parameter λ could be estimated from statistical estimation methods such as a Maximum Likelihood Method by maximizing the logarithm of the likelihood function of the increment of the parameter (Van Noortwijk 2009) and the shape function η_x can be obtained from $\eta_x = \lambda J_x$ in which J_x indicates the average flexural strength deterioration rate associated with the reinforcement corrosion. Assuming J_L as the maximum allowable limit of the structural capacity deterioration, from the definition of probability of failure and by integrating probability density function given in Eq. (7), the lifetime distribution of probability of failure is given by

$$P_f(t) = P_r(J_x \geq J_L) = \int_{J=J_L}^{\infty} f_{J_x}(J)dJ = \frac{\Gamma(\eta_x, J_L\lambda)}{\Gamma(\eta_x)} \tag{8}$$

where $\Gamma(\eta, z) = \int_{v=z}^{\infty} v^{\eta-1}e^{-v}dv$ is the incomplete gamma function for z>0 and η>0. From the aforementioned definition of the failure of corrosion damaged RC structures and Eq. (8), the probability of failure per unit time at the *i*th time interval can then be computed form

$$p_i = P_f(t_i) - P_f(t_{i-1}) \quad \text{for } i = 1, 2, 3 \tag{9}$$

5. Optimised maintenance

In order to evaluate the maintenance strategy of deteriorating RC structures, the estimate of the costs associated with the structural maintenance is essential. The maintenance costs can be obtained by modelling the maintenance of corrosion damaged RC structure as a discrete-time renewal process, whereby renewal process or maintenance actions bring a deteriorating RC structure back into its original condition or as-good-as new state (Van Noortwijk 2009, Chen and Alani 2013). Therefore, in general the cost of maintenance can be defined as the cost which is associated with combination of actions carried out to restore a component or structure to the initial condition. Mathematically a discrete renewal process is a non-negative integer-valued stochastic process that registers the successive renewals in the given time-interval.

Renewal times are considered as the positive, independent, identically distributed, random quantities having the discrete probability function (Van Noortwijk and Frangopol 2004). Because the planned lifetime of most concrete structure is very long, compared with the possible renew cycle length, the strategy for risk cost balanced optimised maintenance during the life time can be approximately considered over an unbounded time horizon. From the renew reward theory and age replacement policy by Van Noortwijk (2009) and Van Noortwijk and Frangopol (2004), the expected cost of repair over an unbounded horizon per unit time depend on the preventive maintenance cost and the corrective maintenance cost, which includes the consequences caused by the failure and the expected renew cycle length, expressed here as

$$C(k) = \frac{\left(\sum_{i=1}^{k} \alpha^i p_i\right) C_F + \alpha^k \left(1 - \sum_{i=1}^{k} p_i\right) C_P}{1 - \left[\left(\sum_{i=1}^{k} \alpha^i p_i\right) + \alpha^k \left(1 - \sum_{i=1}^{k} p_i\right)\right]} \tag{10}$$

where k = 1, 2, 3… represents the number of the age replacement intervals to be determined. The preventive maintenance cost C_P and corrective maintenance cost C_F are the costs associated with preventive and corrective maintenance strategies, respectively. Discount factor α is given by $\alpha=(1+d_r/100)^{-1}$ in which d_r is discount rate per unit time in percentage. Finally, the optimal maintenance time interval k* is evaluated by minimising the expected cost over the life time given in Eq. (10).

6. Worked example

In this section the methodology described above is applied to a numerical example for a simply supported RC beam exposed to an aggressive environment with minimum service life of 50 years. The cross-sectional width and effective depth of beam are b=300 mm and d=560 mm, respectively. Four steel rebar with diameter D_b=16 mm are provided as the tensile reinforcement and two rebar of diameter D_{sc} =12 mm are provided as the compressive steel with clear cover thickness C=40 mm along with the stirrup of diameter D_{st} =6 mm at spacing of 100 mm, subjected to mean annual corrosion current per unit length i_{corr}=6 μA/cm^2. The characteristic compressive strength of concrete is assumed as f_{ck}=40 MPa and the corresponding concrete properties such as tensile strength and modulus of elasticity are estimated from Eurocode 2. The critical and ultimate cohesive crack widths required for this study have been obtained from CEB-FIP model code 1990 for assumed maximum aggregate size of 16 mm. Four cracks are assumed to be formed in the concrete cover, and crack width in the cover concrete is represented by the equivalent crack width, as defined in Khan *et al*. (2014).

Fig. 1 shows the results obtained for residual load capacity predicted by the proposed method for estimating residual flexural strength as a function of the corrosion level, and are compared with the published experiment data of various reference literatures. In Fig. 1, the horizontal axis represents the rebar mass loss (corrosion level) in percentage, and the vertical axis represents the normalized residual load capacity which is calculated by dividing the flexural capacity of corroded element by the capacity of the non-corroded element. As observed in Fig. 1, the flexural strength deterioration predicted by the present study shows the good agreement with the published experimental data of reference literature, such as Almusallam *et al*. (1996), Mangat and Elgarf (1999), EI Maaddawy *et al.* (2005), Torres-Acosta *et al.* (2007), Azad *et al.* (2007), Chung *et al.* (2008), Azad *et al.* (2010), Xia *et al.* (2012), and Zhang *et al.* (2012). At initial corrosion stage, the flexural strength of the corroded beam remains almost the same as that for the un-corroded beam. When corrosion level reaches about 5%, considerable strength deterioration occurs. The reduction in flexural strength is due to the significant reduction in bond strength, which is required to prevent beam from bond failure. At relatively high corrosion level (>5%), bond strength reduction at the steel-concrete interface is the primary factor responsible for the deterioration of flexural strength of the corroded beam rather than the reduction in cross sectional area of the rebar.

The results of probability of failure P_f of the corroded beam are plotted in Fig. 2 for different allowable flexural strength deterioration limits, i.e., J_L= 20%, 25% and 30%, respectively. As discussed earlier, the deterioration of structural performance in terms of structural capacity (flexural strength deterioration) is modelled as a gamma process. As expected, the probability of failure increases with increase in corrosion over time, and significantly depends on the given allowable flexural strength deterioration limit, with a higher probability of failure for a lower allowable limit at the same service time.

In order to find an optimal value of the repair time, the expected cost defined in Eq. (10) is minimised with respect to the number of time interval k. Only relative costs are needed to be considered in calculations. Here, the corrective maintenance cost C_F =1.0 and the preventive maintenance cost C_P = 0.1C_F are adopted in this study. Figure 3 shows the results for the expected relative costs as a function of repair time for various allowable deterioration limits, where the annual discount rate of 5% is considered. The results show that the optimal repair times are 25 year for J_L = 20%, 29 year for J_L = 25%, and 37 year for J_L = 30%, respectively.

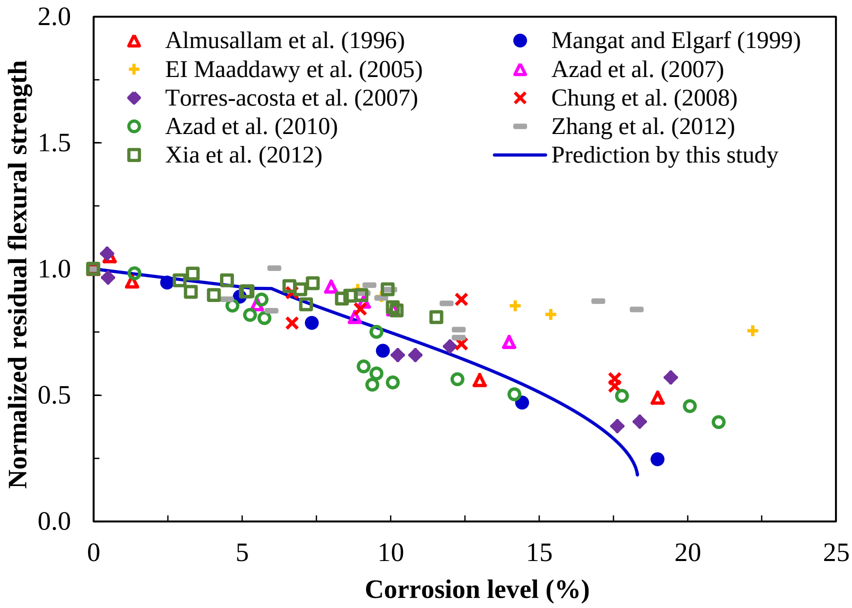

Fig. 1 Flexural strength deterioration versus corrosion level compared with published experimental data available

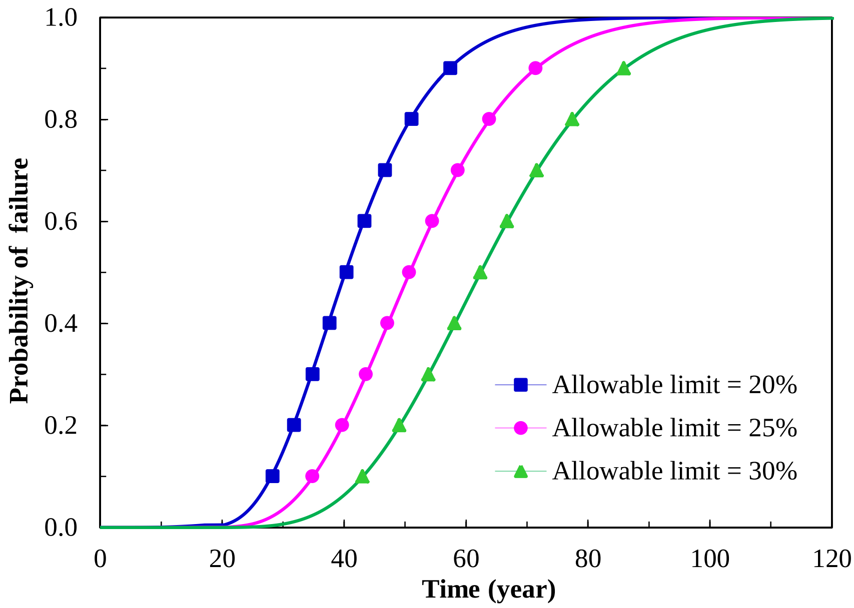

Fig. 2 Probability of structural failure over time for various allowable flexural strength deterioration limits

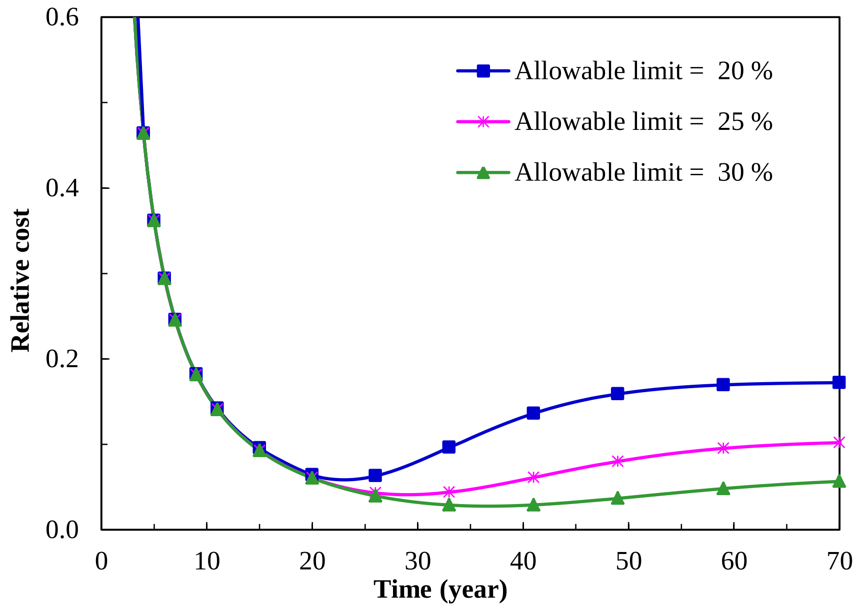

Fig. 3 Expected relative costs over repair time interval with discounting of an annual rate of 5% as a function of repair time for various allowable flexural strength deterioration limits

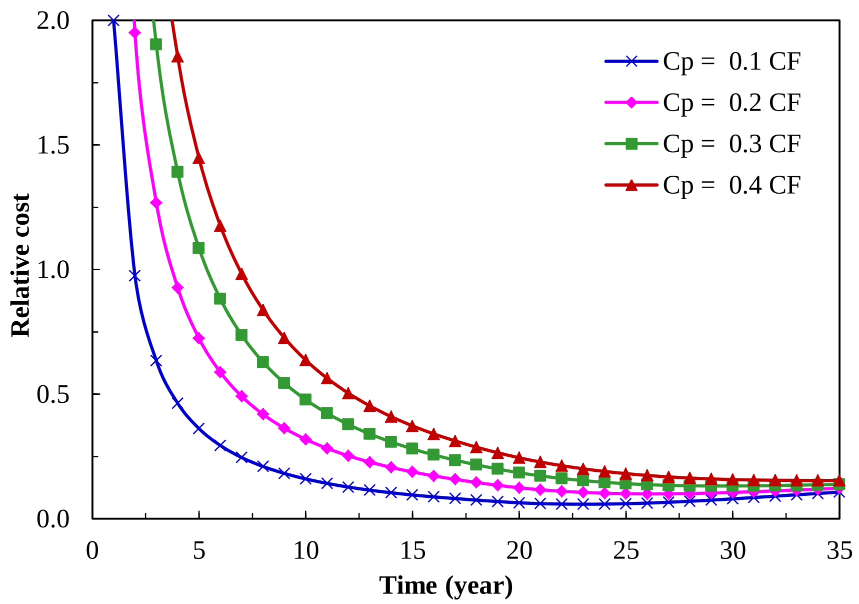

Fig. 4 Expected relative costs as a function of repair time for various preventive maintenance costs C_P in relation to corrective maintenance cost C_F

Fig. 4 shows the influence of the preventive maintenance cost C_P on the optimal repair time, where the preventive maintenance cost ranges from C_P = 10% to 40% of C_F. The allowable flexural strength deterioration limit J_L is set at 20%. It can be seen that the value of the optimal repair time increases when the preventive maintenance cost goes up, from 24 year for $C_P = 0.1C_F$ to 41 year for $C_P = 0.4C_F$. The results also show that earlier repairs are necessary to reduce the risk of failure if the preventive maintenance cost is relatively low.

7. Conclusions

In this study, the structural resistance deterioration caused by reinforcement corrosion is investigated analytically. The results from the proposed method for the flexural strength deterioration due to reinforcement corrosion are then examined by the experimental data available from various sources. A stochastic deterioration model is then employed to evaluate the failure probability of the corroded RC structures by time-dependant reliability analysis during their service life. The optimum repair time is determined by balancing the probability of structural failure and costs for repair. On the basis of the results from the numerical example involving a RC beam subjected to reinforcement corrosion, the following conclusions are drawn: 1) Flexural strength deterioration in the life cycle is significantly affected by reinforcement corrosion, and the proposed approach is capable of predicting the residual strength of corroded RC structure; 2) The proposed stochastic deterioration model based on the gamma process can effectively assess the life cycle flexural strength deterioration with uncertainties for corroded concrete structures; 3) The optimal maintenance strategy during the service life of RC structures affected by reinforcement corrosion can be determined by optimising the balance between the risk of structural failure and the maintenance costs; 4) The optimum structural repair time depends significantly on the chosen allowable flexible strength limit and the ratio of the preventive maintenance cost over the corrective maintenance cost of aging concrete structures.

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