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Load bearing capacity reduction of concrete structures due to reinforcement corrosion

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Abstract. Reinforcement corrosion is one of the major problems in the durability of reinforced concrete structures exposed to aggressive environments. Deterioration caused by reinforcement corrosion reduces the durability and the safety margin of concrete structures, causing excessive costs in managing these structures safely. This paper aims to investigate the effects of reinforcement corrosion on the load bearing capacity deterioration of the corroded reinforced concrete structures. A new analytical method is proposed to predict the crack growth of cover concrete and evaluate the residual strength of concrete structures with corroded reinforcement failing in bond. The structural performance indicators, such as concrete crack growth and flexural strength deterioration rate, are assumed to be a stochastic process for lifetime distribution modelling of structural performance deterioration over time during the life cycle. The Weibull life evolution model is employed for analysing lifetime reliability and estimating remaining useful life of the corroded concrete structures. The results for the worked example show that the proposed approach can provide a reliable method for lifetime performance assessment of the corroded reinforced concrete structures.

Keywords: rebar corrosion; concrete cracking; strength deterioration; lifetime reliability; residual life

1. Introduction

Reinforcement corrosion is a global problem in the durability of the reinforced concrete (RC) structures. It causes considerable cost and safety threats to civil engineering infrastructure. Reinforcement (rebar) corrosion affects the performance of corroded RC structures in different ways. Structural performance deterioration mainly depends on the loss of rebar area, cracking in concrete cover and bond strength degradation between the rebar and concrete. Corrosion progress in concrete structures affects the mechanical properties of both concrete and reinforcement. These changes in mechanical properties along with decreasing size of the rebar and increasing crack width in the concrete cover can lead to significant reduction in the residual load carrying capacity and stiffness of RC structures (Azad 2010, Chen 2018a). As corrosion progresses, this internal stress becomes greater than the tensile strength of the concrete, radial splitting cracks initiates at the rebar surface and propagates towards the cover surface. When corrosion continues, it may lead towards the eventual spalling and delamination of the concrete cover. This ultimately reduces the bond strength between the rebar and concrete and load carrying capacity, causing the structure failure (Lounis *et al*. 2006), as shown in Fig. 1.

Many investigations have been undertaken during the last three decades regarding the prediction of corrosion

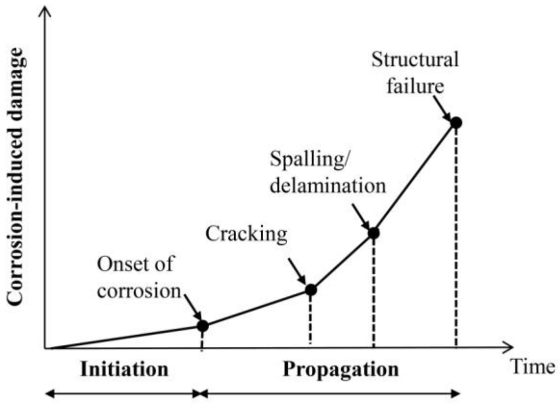

Fig. 1 Corrosion induced damage in corroded RC structures

initiation, but comparatively few studies have been undertaken in corrosion propagation and even less with the residual structural capacity of the corroded RC structure (Torres-Acosta *et al*. 2007). Limited research has been carried out to investigate the effect of reinforcement corrosion on the mechanical characteristics and load carrying capacity of corroded RC structures, where reinforcing bars were corroded by using accelerated corrosion technique (Chen 2018b, Vidal *et al*. 2004, Zhang *et al*. 2010). Also, research has been undertaken during last decade regarding the influence of reinforcement corrosion and concrete cracking on the performance of reinforced concrete structures (Bhargava *et al*. 2006, Ortega *et al*. 2018) and to predict the residual life of structures (Torres-Acosta *et al*. 2007, Chen and Nepal 2016). The time-

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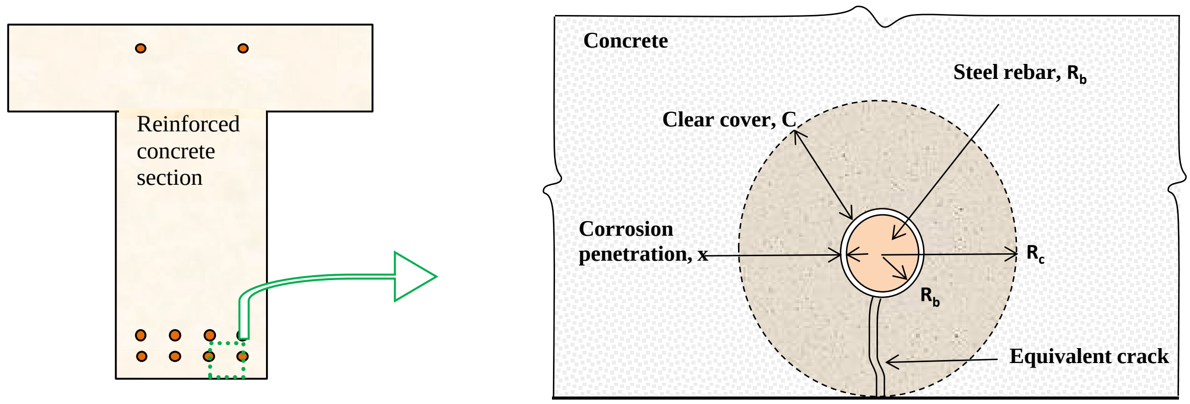

(a) T-girder cross section (b) Modelling of cover concrete

Fig. 1 Thick-walled cylinder model adopted for idealising cover concrete of a RC structure

dependent structural reliability has been utilised to evaluate strength degradation and performance deterioration over time during service life (Stewart and Suo 2009, Chen and Xiao 2015). Recently some theoretical approaches have been proposed by Chen (2018a) and Chen and Nepal (2018). However, the effect of concrete cracking on residual load carrying capacity and remaining useful life of corroded structures is not well understood. There is a need of a reliable performance assessment method which can confidently estimate the remaining useful life and predict the residual load carrying capacity of concrete structures subjected to reinforcement corrosion.

The paper presents an approach for lifetime performance assessment of corrosion damaged reinforced concrete structures on the basis of the lifetime distribution modelling of concrete crack width evolution and structural strength deterioration. The model for concrete crack growth due to corrosion progress is proposed, which is used to investigate its effects on rebar residual bond strength. Many factors, such as rebar cross-section loss and bond strength degradation, are taken account for estimating the load bearing capacity deterioration of the corroded concrete structures. The predicted concrete crack width and structural strength deterioration are then utilised as random variables for lifetime distribution analyses. Based on the Weibull life distribution model for the concrete cracking evolution and flexural strength deterioration, the lifetime reliability is predicted from usage, and the remaining useful life is estimated by the current state of the structure.

2. Cover concrete cracking

Initially, in reinforced concrete structures, the steel rebar embedded in concrete is naturally protected from corrosion by the strong alkalinity of the concrete. The ingress of chlorides through the concrete cover deactivates the natural protective oxide layer formed around the reinforcements. Once the protective layer is disturbed in presence of enough oxygen and moisture content, corrosion is initiated. Depending on the level of oxidation, corrosion product may expand by as much as six times to its original volume. Thus, the reinforcement corrosion consumes the original steel and causes the loss of steel cross-section and mass. The average corrosion penetration along the rebar perimeter over time t to account for non-uniform corrosion is defined as

$$x(t) = \frac{\Delta A_s(t)}{\pi\phi_0} \tag{1}$$

where ϕ_0 is initial rebar diameter, and ΔA_s is the loss of steel cross-sectional area caused by uniform corrosion or non-uniform corrosion such as pitting, calculated from

$$\Delta A_s(t) = \frac{1}{\rho_r \xi_r}\sqrt{m_c \pi \phi_0 i_{corr} t} \tag{2}$$

in which ρ_r is density of corrosion rust with an approximate value of $\rho_r = 3600 kg/m^3$; ξ_r is volume expansion factor measured from experiments; m_c is an empirical coefficient taken as $m_c = 2.1 \times 10^{-2}$; i_{corr} represents the mean annual corrosion current per unit length at the surface area of the rebar (A/m^2).

The corrosion penetration rate over time, defined as the average corrosion penetration over the radius of original steel rebar, is expressed by

$$\chi(t) = \frac{x(t)}{\frac{1}{2}\phi_0} \tag{3}$$

To accommodate the volume increase per unit length caused by reinforcement corrosion product, the uniform displacement over time at the interface between the steel rebar and the surrounding concrete due to corrosion penetration is calculated from

$$u_b(t) = (\xi_r - 1)x(t) \tag{4}$$

The prescribed displacement $u_b(t)$ due to reinforcement corrosion generates cracking in cover

Table 1 Empirical models for estimating crack initiation and crack growth

Reference	Crack initiation	Crack growth
Alonso *et al*. (1998)	$x_c(\mu m) = 7.53 + 9.3\frac{C}{D_b}$	/
Rodriguez *et al*. (1996)	$x_c(\mu m) = 83.8 + 7.4\frac{C}{D_b} - 22.6 f_t(MPa)$	$w(mm) = 0.05 + \beta(x - x_c)$
Webster and Clark (2000)	$x_c(\mu m) = 1.25C(mm)$	/
Vidal *et al*. (2004)	$\Delta A_{so}(mm^2) = A_s[1-[1-\frac{\alpha_p}{D_b}(7.53+9.32\frac{C}{D_b})10^{-3}]^2]$	$w(mm) = 0.0575(\Delta A_s - \Delta A_{so})$
Zhang *et al*. (2010)	/	$w(mm) = 0.1916\Delta A_s + 0.164$

The concrete cracking process can be investigated by taking a single steel rebar and the surrounding concrete and considering thick walled cylinder model, as shown in Fig. 1.

From extensive experimental studies, a linear relationship between the corrosion level and crack width has been observed. On the basis of the experimental observation, a number of formulas relating the crack width with the amount of corrosion or sectional loss of rebar have been proposed. Some of the empirical models proposed by these experimental studies are summarised in Table 1.

In Table 1, x_c and ΔA_{so} are the corrosion penetration and cross-section loss associated with visible crack development at cover surface; w is crack width and ΔA_s is the sectional loss of steel rebar of a diameter D_b and a concrete cover thickness C. These empirical relations are obtained from the regression analysis of the experimental results obtained, and therefore these empirical models usually cannot take into account all of the relative parameters. Therefore, the analytical methods which are capable to consider more relevant parameters, such as concrete properties, are needed to study corrosion-induced cracking process.

In the analytical models recently developed by Chen (2018a), concrete cracking can be modelled as a process of tensile softening if cracking is considered as cohesive. The actual crack width over time at the concrete surface $w_c(t)$, which is predicted from the he idealised thick-walled cylinder model, is proportional to the average corrosion penetration, expressed here as

$$w_c(t) = \eta_w W_u \frac{G_f}{f_{ct}}\left(\frac{E_c}{f_{ct}}(\xi_r - 1)x(t) - \eta_c\right) \quad (5)$$

where W_u is the normalised value of the ultimate cohesive crack width w_u when residual tensile strength vanishes; G_F is the fracture energy of concrete; E_c is the modulus of elasticity for intact concrete; f_{ct} is the tensile strength of intact concrete. The coefficients η_w and η_c are defined, respectively, as

$$\eta_w = \frac{1}{(l_0 - R_b)[1 - R_c(l_0 - R_c)\delta(R_c, R_b)]}$$
$$\eta_c = R_b + R_c(l_0 - R_b)(l_0 - R_c)\delta(R_c, R_b) \quad (6)$$

in which $R_b = \varphi_0/2R_b = \varphi_0/2$ and $R_c = C + \varphi_0/2$, as shown in Fig. 1; $l_0 = n_c W_u l_{cr}/2\pi$ where l_{ch} is characteristic length defined in Bažant and Planas (1998) as $l_{ch} = E_c G_F/f_{ct}{}^2$; and the total number of the cracks in the cover n_c is estimated from $n_c = 2\pi R_c/L_c$ in which the spacing of crack bands L_c is approximately three times the maximum aggregate size. The crack width coefficient δ (R_c, R_b) is defined as

$$\delta(R_c, R_b) = \frac{(R_c - R_b)}{l_0(l_0 - R_c)(l_0 - R_b)} + \frac{1}{l_0{}^2}\ln\frac{R_c|l_0 - R_b|}{R_b|l_0 - R_c|} \quad (7)$$

The corrosion penetration (x_c) required for extending the cracks throughout the cover concrete with Poisson's ratio v is given by

$$x_c = \frac{f_{ct}}{E_c}\frac{\xi_r}{(\xi_r - 1)}[R_b + (1+\nu)R_c(l_0 - R_b)(l_0 - R_c)\delta(R_b, R_c)] \quad (8)$$

The above analytical predictions of the corrosion penetration required to appear cracks at the concrete cover surface are compared with experimental data observed by Vu *et al*. (2005) and Andrade *et al*. (1993), as shown in Table 2. From the results, the predicted time-to-crack corrosion penetration (x_c) in general agree with the experimental results. Some discrepancies in the results may be due to complexity of the cracking process.

Finally, the ultimate corrosion penetration (x_u) required for cracks to reach the ultimate cohesive value is given by

$$x_u = \frac{f_{ct}}{E_c}\frac{l_0}{(\xi_r - 1)} \quad (9)$$

After cracks reach the ultimate cohesive value, the width of the cracks in the cover concrete still increases due to the progressive expansion of the rust layer. Here, the equivalent crack width $w = n_c w_c$, defined as the cumulated crack width over the cover, is utilised for evaluating rebar strength degradation of the concrete structures.

3. Residual load carrying capacity

The rebar bond is the interaction mechanism that enables the force transfer between rebar and the surrounding concrete. Without the bond, composite action

Table 2 Comparison for experimental and predicted time-to-crack corrosion penetrations (x_c)

Reference	Cover thickness, C (mm)	Rebar diameter, D_b (mm)	Compressive strength, f_c (MPa)	Tensile strength, f_{ct} (MPa)	Observed, x_c (%)	Predicted, x_c (%)
Vu *et al*. (2005)	25	16	52.7	4.55	0.74	0.66
	50	16	—	—	1.62	1.45
	25	16	20	3.06	0.44	0.43
	50	16	—	—	1.33	1.32
Andrade *et al*. (1993)	20	16	—	3.55	0.36	0.38
	30	16	—	3.55	0.53	0.61

in RC structures cannot occur. When composite action is disrupted, load carrying capacity (flexural capacity) of RC structures is also disrupted. Here, the bond degradation due to reinforcement corrosion is considered in evaluating the load carrying capacity deterioration of RC structures.

From the analytical and experimental investigations by Giuriani *et al*. (1991) and by ignoring the confining actions, the bond strength deterioration over time $\tau(t)$ depends on concrete crack width, simplified here as

$$\tau(t) = \frac{1}{1+\lambda_\eta\, w(t)/\phi_0}\tau_{u0} \tag{10}$$

where τ_{u0} is the ultimate bond strength of the rebar with intact concrete, estimated from design codes (CEB-FIP 1990), and λ_η is the coefficient associated with concrete properties but independent of φ_0 and determined by experiments.

To consider the effect of the bond strength degradation in evaluating flexural strength of corroded RC structures, a simply supported RC beam subjected to flexural load is now considered, as shown in Fig. 2(a). Meanwhile, Figures 2(b) and 2(c) show the parameter definitions and the strain and stress distribution across beam section under initial un-corroded condition of reinforcing bars as given by Eurocode 2 (2004), respectively.

For the corroded RC beam, when ultimate bond strength is insufficient to prevent anchorage failure, the tensile force generated in the corroded tensile steel can be obtained from

$$F_{sx} = n_b \pi \phi_r l_d \tau_{ux} \tag{11}$$

where n_b is the number of the tensile steel bars at the bottom of the beam section; $\varphi_r(t) = \varphi_0 - 2x(t)$ is residual rebar diameter at time t due to reinforcement corrosion; l_d is the development length evaluated from design code (CEB-FIP 1990); and τ_{ux} is the ultimate bond strength of corroded rebar at average corrosion penetration x.

The strain compatibility of a RC beam with corroded reinforcement can be considered between un-bonded and bonded condition (Chen and Nepal, 2018). By 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

$$\frac{\varepsilon_{stx}}{\varepsilon_{ccx}} = g_x \frac{d-y_x}{y_x} \quad \text{and} \quad \frac{\varepsilon_{scx}}{\varepsilon_{ccx}} = g_x \frac{y_x - d'}{y_x} \tag{12}$$

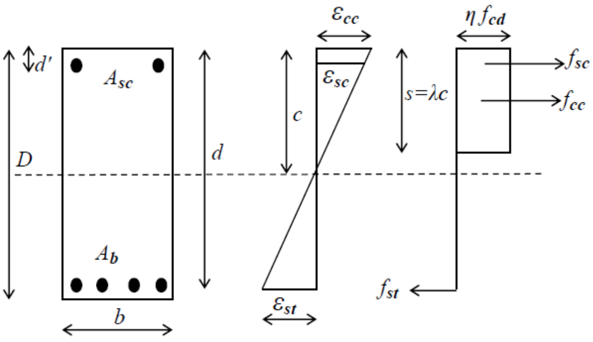

Fig. 2 Flexural analysis of a RC beam section: (a) typical RC beam section; (b) strain distribution; (c) equivalent stress distribution

where 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 is the effective depth of beam; and d' is the distance from the centroid of the compression steel to the edge of the compressive fibre; and g_x is the interpolation factor, which can be obtained by considering the bond strength value of perfectly bonded and un-bonded conditions of the RC beam, expressed here as

$$g_x = 1-(1-\frac{\tau_{ux}}{\tau_{u0}})(1-\frac{l_{eq}}{l_d}) \tag{13}$$

in which plastic equivalent region $l_{eq} = 9.3y_x$; and l_d is the ultimate development length required to prevent anchorage (bond) failure of the tensile steel rebar, estimated from Eurocode 2 (2004).

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 then be evaluated by considering the failure modes of flexural strain at the compressive fibre and tensile fibre. These failure modes can be determined by satisfying the limiting values of ε_{stx}, ε_{ccx} and ε_{scx}, as given in Eurocode 2 (2004). Finally, by taking moment at the centroid of the tensile steel, the residual flexural strength of the corroded RC beam can be calculated from

$$M_{ux} = F_{cx}(d - 0.5\lambda' y_x) + F_{sx}(d-d') \tag{14}$$

where the compressive force of the concrete in compression F_{cx} is determined by

$$F_{cx} = \eta \lambda' y_x b f_{cd} \tag{15}$$

where $f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$ is the design compressive strength of the concrete in which α_{cc} is the constant taken as 0.85, f_{ck} is the characteristic compressive strength of the concrete (assuming here $f_{ck} \leq$ 50MPa) and γ_c is taken as 1.0 in estimating the ultimate flexural strength of existing concrete structures; b and h are width and overall depth of the cross-section of the beam, respectively; s is the equivalent compressive zone of concrete given by $s = \lambda' y_x$; η and λ' are the coefficients taken as 1 and 0.8, respectively (CEB-FIP 1990).

4. Lifetime reliability analysis

Lifetime distributions are often used to assess the performance state of a system over the life cycle through lifetime reliability that represents the time to failure as continuous and non-negative random variables (Chen and Xiao 2015, Chen and Zhang 2017). The lifetime distributions reflect the combined effect of the contributions from all uncertainties on the system, including loads and resistance. Lifetime reliability is an appropriate life distribution for threshold-based approaches and is defined as the probability of the system to be functional at a given time instant. For a system with failure time T_s, the lifetime reliability $R(t)$ of the system with non-repair can be expressed as

$$R(t) = P[t \leq T_s] \tag{16}$$

For the corroded reinforced concrete beams, the corrosion-induced concrete crack width and flexural strength can be chosen as the random variables for life distribution analysis. The lifetime evolution of the chosen random variables is obtained from the predictions discussed previously. It is well known that the Weibull model is very flexible life distribution model and widely used for undertaking lifetime reliability analysis and performance predictions (Barone and Frangopol 2014). In this study, the Weibull life distribution model is adopted for the evolution of random variables (S), such as corrosion-induced concrete crack width and flexural strength, defined as

$$\frac{S}{S_b} = [-\ln(1 - \frac{\bar{x}}{\bar{x}_b})]^{1/\beta} \tag{17}$$

where $S_b = \upsilon S_l$ and υ is the scale factor that can be determined by design requirements with respect to allowable limit S_l; $\bar{x}$ and $\bar{x}_b$ are average corrosion penetration and that corresponding to S_b, respectively; $\beta > 0$ is the shape parameter to be determined by fitting the predicted results to the Weibull model, estimated from

$$\beta = \frac{\sum_i \{\ln[-\ln(1 - \frac{\bar{x}_i}{\bar{x}_b})]\}^2}{\sum_i \{\ln[-\ln(1 - \frac{\bar{x}_i}{\bar{x}_b})]\} \ln(\frac{S_i}{S_b})} \tag{18}$$

Considering the serviceability requirements of a corroded concrete structure, the crack width at the concrete surface (w) is now selected as the Weibull random variable, namely

$$S = w(\bar{x}(t)) \quad \text{and} \quad S_b = \upsilon w_l \tag{19}$$

where w_l is allowable crack width for the serviceability requirements.

Due to the lack of experimental data available for the performance deterioration of actual corroded concrete structures, it is assumed here that the times to failure of the corroded concrete structures follow the Weibull distribution. For ultimate limit state requirements, the flexural strength deterioration rate of the corroded concrete beam is chosen as the Weibull random variable, since the value of deterioration rate is non-negative and continuous over time, namely

$$S = (M_{u0} - M_{ux}(\bar{x}(t)) / M_{u0} \quad \text{and} \quad S_b = \upsilon (M_{u0} - M_{ul}) / M_{u0} \tag{20}$$

where M_{uo} is original flexural strength of the intact concrete structures, and M_{ul} is allowable flexural strength for the ultimate limit state requirements.

For the chosen Weibull random variable (S), and the probability density function is written here as

$$f(S) = \frac{\beta}{S} (\frac{S}{S_b})^\beta \exp[-(\frac{S}{S_b})^\beta] \tag{21}$$

then the probability of failure (cumulative distribution function) as a function of average corrosion penetration over time is calculated from

$$p_f(\bar{x}(t)) = 1 - \exp[-(\frac{S(\bar{x}(t))}{S_b})^\beta] \tag{22}$$

From the probability of failure, the lifetime reliability for the chosen Weibull random variable as a function of corrosion penetration over time is given as

$$R(\bar{x}(t)) = \exp[-(\frac{S(\bar{x}(t))}{S_b})^\beta] \tag{23}$$

The residual capacity for the chosen Weibull random variable, when the variable reaches the value of S_m at time T_m, can be expressed as

$$S_r = \int_{S_m}^{\infty} R(S) dS \tag{24}$$

Consequently, the remaining useful life T_{rul} associated with the chosen Weibull random variable of the corroded concrete structure that is still surviving at service age of T_m is estimated from

$$T_{rul} = \int_{T_m}^{\infty} R(\bar{x}(t)) dt \tag{25}$$

The remaining useful life T_{rul} depends on the given allowable limit for the chosen Weibull random variable of the corroded concrete structure, i.e. corrosion-induced concrete crack width and flexural strength deterioration rate.

Due to inevitable variation in the limited experimental data available for corrosion-induced concrete cracking, a confidence interval is introduced here to represent the uncertainty in the estimates of concrete crack width growth and the associated lifetime reliability and remaining useful life. The confidence interval specifies a range within which the parameter is estimated to lie. A confidence interval with a confidence level of [100(1-γ)]% (Soong 2004), where γ ranges from 0 to 1 related to the given confidence level, for the predicted concrete crack growth w in terms of corrosion penetration rate χ, can be determined by

$$L_{1,2} = w \mp t_{n-2,\gamma/2}\left\{\Sigma^2\left[\frac{1}{n}+(\chi_i-\bar{\chi})^2\left(\sum_{i=1}^{n}(\chi_i-\bar{\chi})^2\right)^{-1}\right]\right\}^{1/2} \quad (26)$$

where $L_{1,2}$ represents the lower and upper limits of the confidence interval, respectively; $t_{n-2,\gamma/2}$ is t-distribution with (n-2) degrees of freedom and probability of $\gamma/2$; n is the total number of test data available; χ_i and $\bar{\chi}$ are the test data and their mean value of corrosion penetration rate; Σ is the unbiased estimator for variance and calculated from

$$\Sigma^2 = \frac{1}{n-2}\left[\sum_{i=1}^{n}(w_i-\bar{w})^2 - \beta^2\sum_{i=1}^{n}(\chi_i-\bar{\chi})^2\right] \quad (27)$$

in which w_i and $\bar{w}$ are the test data and their mean value of concrete crack width; and coefficient β is determined from

$$\beta = \left[\sum_{i=1}^{n}(\chi_i-\bar{\chi})(w_i-\bar{w})\right]\left[\sum_{i=1}^{n}(\chi_i-\bar{\chi})^2\right]^{-1} \quad (28)$$

From the obtained confidence interval for the concrete cracking, the confidence intervals for the associated lifetime reliability and remaining useful life can be estimated.

5. Numerical example

A reinforced concrete T-girder beam, as shown in Fig. 1(a), is now used to demonstrate the applicability of the proposed method. The corroded concrete T-girder cross section has a dimension of height h=500mm and widths b=600mm at the top and b=250mm at the bottom. Eight reinforcing steel bars of a diameter of 16mm are embedded into the bottom of the concrete T-girder section (tensile zone), and identical steel bars are placed in the top (compressive zone), with an average clear cover thickness of 40mm. The reinforcing steel has a tensile strength of f_{yo} = 460MPa and modulus of elasticity E_{st} = 200GPa. The concrete has a specified compressive strength f_c=35MPa, concrete tensile strength $f_{ct} = 0.69\sqrt{f_c} = 4.08MPa$, and modulus of elasticity E_c= 4400$f_c^{0.516}$ = 27.6 GPa (Stewart and Rosowsky 1998). Concrete fracture energy G_F = 96N/m and the ultimate cohesive crack width w_u = 0.15mm are estimated from the given compressive strength where the maximum aggregate size is assumed to be 25mm. The total

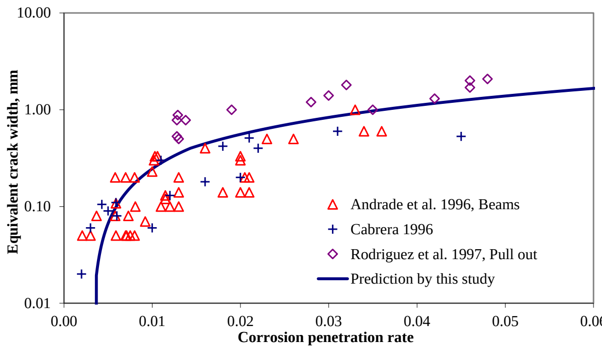

Fig. 3 Predicted equivalent concrete crack width growth with corrosion penetration rate, compared with experimental results in Torres-Acosta and Martinez-Madrid (2003)

crack number n_c is estimated from $n_c = 2\pi R_c/L_c \approx 4$. Here, a corrosion level of 1.0$\mu A/cm^2$ for the medium corrosive environment is assumed for calculating the corrosion penetration rate.

Figure 3 gives the results for the equivalent crack width development in cover concrete as a function of the corrosion penetration rate, which is predicted from $w = n_c w_c$ and Eq. (5). The predicted results are then compared with the experimental data obtained from various sources by various corrosion test procedures for corroded reinforced concrete structures (Torres-Acosta and Martinez-Madrid 2003). Here, the corrosion penetration rate is adopted in the comparisons, since there is no need for the corrosion density and the corrosion time that may be different in various tests as well as in the predictions. The predicted results for the corrosion-induced crack growth match well the experimental data available.

Figure 4 shows the results for various confidence intervals for the predicted concrete cover crack width growth as a function of corrosion penetration rate. Here, the 90% and 99% confidence bands for the predicted concrete crack width growth are determined from the experimental data available from various sources, and the individual confidence band consists of the space between the corresponding upper and lower limit curves. The test data shown in Fig. 3 are used for determining the confidence intervals, and the estimated unbiased standard deviation is 0.43. The results indicate that there is a 90% or 99% probability that the true concrete crack width growth will lie within the corresponding confidence interval calculated from the experimental data.

The lifetime evolution of corrosion-induced concrete cracking is modelled by the Weibull lifetime distribution discussed in Section 4 in detail, where the crack width on the cover surface is chosen as the Weibull random variable for the corroded concrete girder during the service life, as shown in Fig. 5. Different values for the allowable crack width limit, i.e. 0.3mm, 0.4mm and 0.5mm, are adopted for the lifetime distribution as a function of corrosion penetration rate and for investigating its effect on lifetime reliability. The shape parameter of the Weibull model γ =2.6 is determined from the validated predictions. As expected, as corrosion penetration rate increases, the lifetime

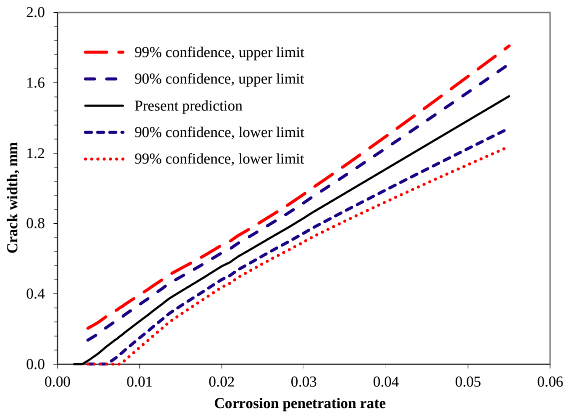

Fig. 4 Confidence intervals for predicted concrete crack width evolution with 90% and 99% confidence levels

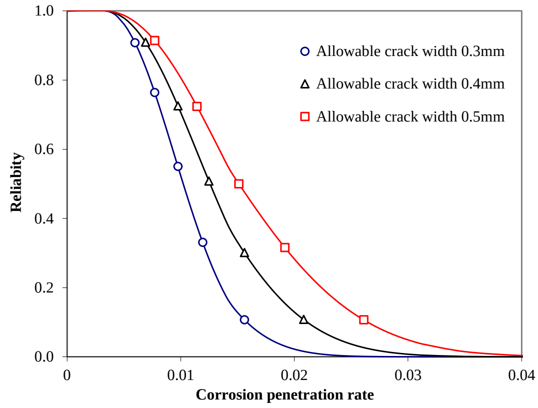

Fig. 5 Lifetime reliability associated with concrete cracking evolution for various allowable crack width limits

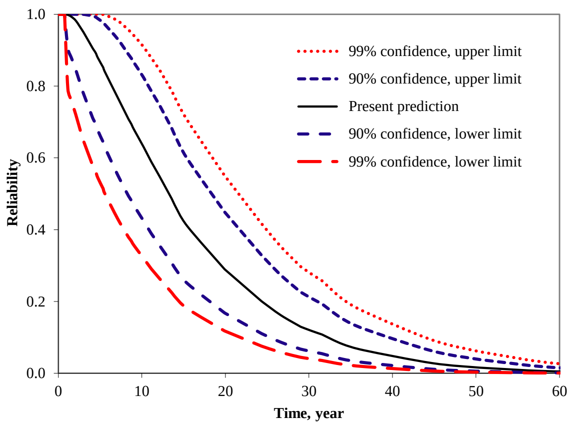

Fig. 6 Predicted lifetime reliability associated with concrete cracking evolution

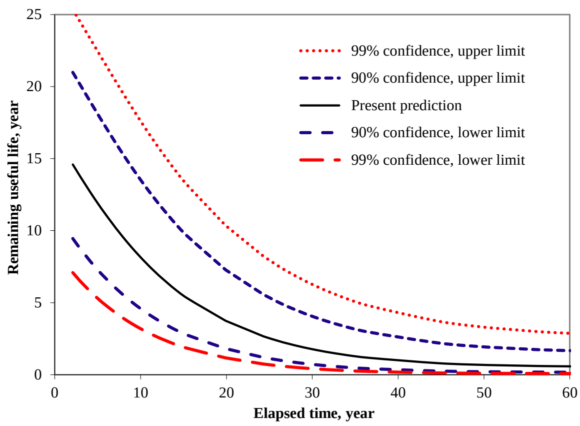

Fig. 7 Predicted remaining useful life over time due to concrete cracking

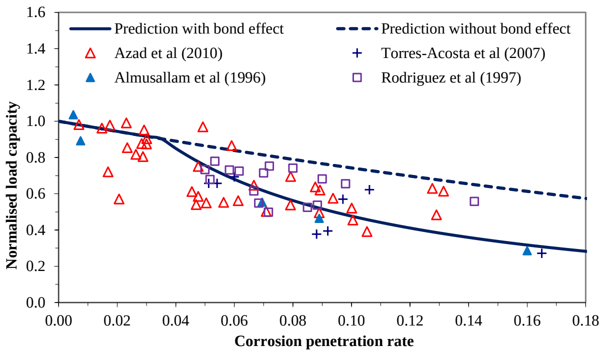

Fig. 8 Predictions of the normalised residual flexural load bearing capacity evolution, compared with experimental results from various sources

reliability associated with concrete cracking evolution of the corroded concrete girder decreases. Also, the allowable crack width limit for the corroded concrete girder has significant influence on the lifetime reliability of the structure. The lifetime reliability increases when the value of allowable crack width limit becomes higher.

The results for lifetime reliability over time associated with concrete cracking for various confidence intervals are plotted in Fig. 6. Here, the allowable crack width limit of 0.4*mm* is adopted for the lifetime reliability analysis, and the corrosion level of 1.0$\mu A/cm^2$ is assumed for the medium corrosive environment. The confidence intervals for the predicted lifetime reliability due to corrosion-induced concrete cracking are determined on the basis of the results from the obtained confidence intervals given in Fig. 4, and then are provided for probabilities of 90% and 99% in Fig. 6, respectively. The lifetime reliability decreases with time due to the growth of corrosion-induced concrete cracking.

Figure 7 show the results for the predicted remaining useful life over time due to corrosion-induced concrete cracking with 90% and 99% confidence intervals on the basis of the results given in Fig. 6. The remaining useful life for the corroded reinforced concrete girder is predicted from the estimated remaining serviceability related to concrete crack growth, where the allowable crack width limit of 0.4*mm* and the corrosion level of 1.0$\mu A/cm^2$ are considered. From the obtained results, the remaining useful life largely depends on the age of the corroded concrete girder in service and the current value of the crack width on concrete surface. Here, the remaining useful life represents the expected remaining service life of the corroded concrete

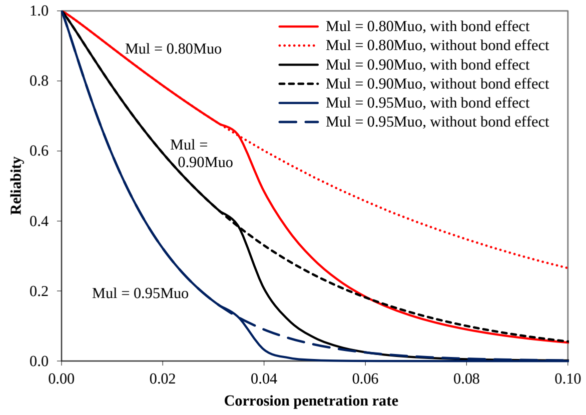

Fig. 9 Lifetime reliability associated with flexural strength deterioration for various allowable flexural strength limits

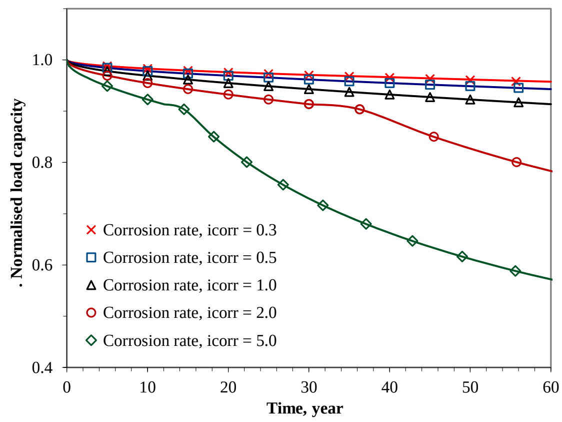

Fig. 10 Normalised residual flexural load bearing capacity evolution over time for various assumed corrosion levels

girder at a given time instant, under the condition that the concrete crack width does not exceed the predefined allowable limit for serviceability requirements at the given time.

Figure 8 shows the results for the residual flexural strength of the corroded concrete girder as a function of corrosion penetration rate. Two cases are considered here, i.e. the residual flexural capacity is estimated with and without consideration of rebar bond strength degradation, respectively. The results predicted by the proposed method with consideration of bond effect are in a better agreement with the experimental data, compared with results for the case without bond effect. From the results, the failure mode of the corroded concrete girder changes from rebar yielding failure at the early corrosion stage to rebar anchorage failure at corrosion penetration rate of approximately 0.04, where the residual flexural strength has a sharp drop. Thus, the rebar bond strength degradation should be considered in determining the residual flexural load bearing capacity after the critical time for the change of failure mode.

From the predicted flexural strength deterioration, the lifetime reliability associated with ultimate moment resistance deterioration is obtained for various allowable flexural strength limits M_{ul}, as shown in Fig. 9. The allowable flexural strength limits represent the thresholds of the flexural strength for the ultimate limit state requirements, and three cases with the allowable flexural strength limits of M_{ul}=0.80M_{uo}, 0.90M_{uo} and 0.95M_{uo} are considered. From the obtained results, the associated lifetime reliability affected by reinforcement corrosion decreases steadily with corrosion penetration rate. Here again, the lifetime reliability reduces sharply after the critical time at corrosion penetration rate of approximately 0.04. The associated lifetime reliability is significantly affected by allowable flexural strength deterioration limit.

In order to investigate the influence of reinforcement corrosion level on the residual flexural strength of the corroded concrete structure, various values of corrosion level are assumed here. The results in Fig. 10 show the influence of reinforcement corrosion level on the residual flexural strength, where the value of corrosion level ranging from 0.3$\mu A/cm^2$ to 5.0$\mu A/cm^2$ is considered. As expected, the corrosion level significantly affects the deterioration of the ultimate moment resistance of the corroded concrete girder. Furthermore, the failure mode of the structure can be changed from rebar yielding failure to rebar anchorage failure at much earlier stage in the case with higher corrosion level.

Figure 11 shows the results for lifetime reliability associated with ultimate moment resistance deterioration over time, where corrosion level ranging from 0.3 to 5.0$\mu A/cm^2$ and allowable flexural strength limit of M_{ul}=0.90M_{uo} are considered. Due to rebar cross-section loss, yielding strength reduction and bond strength degradation, the associated lifetime reliability decreases steadily with time. The rebar anchorage failure mode dominates the ultimate moment resistance of the corroded concrete girder at the service time of approximately 15*year* and 37*year* for the cases with corrosion levels of 5.0$\mu A/cm^2$ and 2.0$\mu A/cm^2$, respectively. Here again, the higher corrosion level at the rebar surface causes lower lifetime reliability associated with flexural strength of the structure.

From the obtained lifetime reliability, the remaining useful life associated with the flexural resistance deterioration of the corroded concrete girder can be predicted, as shown in Fig. 12, where the allowable flexural strength limit of M_{ul}=0.90M_{uo} is considered. As expected, the residual useful life associated with ultimate moment resistance deterioration largely depends on reinforcement corrosion level. The remaining life decreases as reinforcement corrosion level increases, reducing to 5.6*year* if the concrete structure is still surviving after 60 years' service in the case with corrosion level of 2.0$\mu A/cm^2$.

6. Conclusions

This paper presents an effective approach for evaluating the load carrying capacity deterioration of corrosion affected RC structures. The application of the proposed approach is illustrated with the numerical example. On the basis of the results obtained from the numerical example, the following conclusions can be drawn: 1) The proposed approach is capable of evaluating the crack growth and

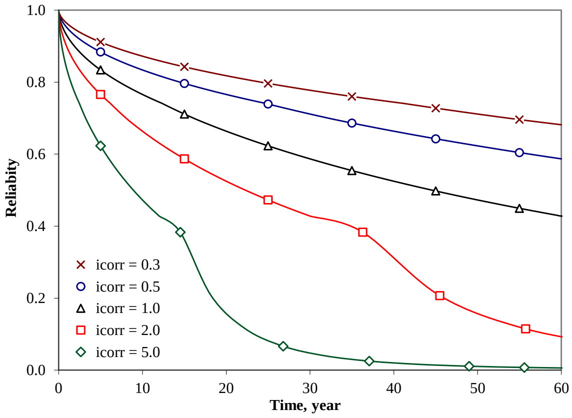

Fig. 11 Lifetime reliability associated with flexural strength deterioration for various assumed corrosion levels

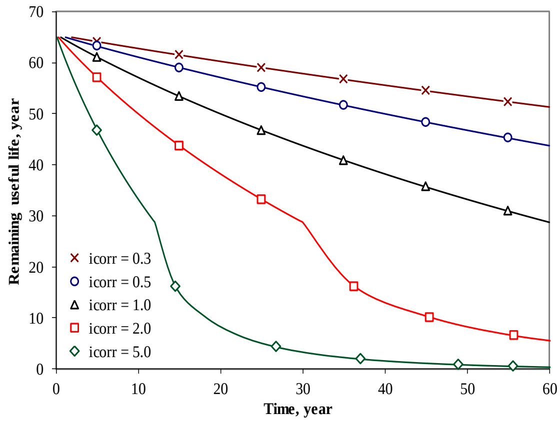

Fig. 12 Remaining useful life due to flexural strength deterioration for various assumed corrosion levels

residual load carrying capacity deterioration of the corroded RC structures, and the proposed analytical model provides a linear relationship between the crack width and the corrosion penetration; 2) The concrete crack width and flexural strength deterioration due to reinforcement corrosion are key performance indicators, thus can be chosen as random variables for lifetime distribution analyses and structural performance assessment; 3) Flexural strength decreases significantly after the critical time at corrosion penetration rate of approximately 0.04, which is associated with significant reduction in bond strength and the corresponding anchorage failure occurred before yielding of steel rebar and surrounding concrete; 4) The Weibull model is an appropriate lifetime distribution for evaluating lifetime reliability and estimating remaining useful life, which is useful for further determining the optimal condition-based maintenance strategy.

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