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Gamma process modelling for lifecycle performance assessment of corrosion affected concrete structures

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ABSTRACT

Life cycle performance of corrosion affected RC structures is an important and challenging issue for effective infrastructure management. The accurate condition assessment of corroded RC structures mainly depends on the effective evaluation of deterioration occurring in the structures. Structural performance deterioration caused by reinforcement corrosion is a complex phenomenon which is generally uncertain and non-decreasing. Therefore, a stochastic modelling such as the gamma process can be an effective tool to consider the temporal uncertainty associated with performance deterioration. This paper presents a time-dependent reliability analysis of corrosion affected RC structures associated with the bond strength degradation. Initially, analytical solutions are provided to evaluate the crack width at the cover surface and predict the corresponding loss of bond strength between the corroded steel and the surrounding cracked concrete. Then in order to model the progression of bond strength deterioration during the life cycle of the RC structure, a gamma process model is adopted. The time-dependent reliability analysis is then applied to evaluate the probability of failure of the RC structure. Finally, a numerical example is used to demonstrate the applicability of the proposed approach. The results from the illustrative example show that the proposed approach is capable of assessing performance of the bond strength of concrete structures affected by reinforcement corrosion during their lifecycle.

1. INTRODUCTION

The performance of the reinforced concrete (RC) structures exposed to aggressive environments such as motorway bridges, car parks and marine structures is often seriously affected by corrosion in reinforcement. The corrosion products formed during corrosion process are expansive in nature (Pantazopoulou and Papoulia 2001). This volume increase creates expansive force at the rebar surface and subsequently produces radial splitting cracks in the surrounding concrete. Furthermore, the loss of cross-sectional area of the rebar and the loss of bond strength between corroded steel and concrete directly affect the flexural strength of the RC structure (Bhargava et al. 2007a, Shetty et al. 2014). As a result, their performance is compromised.

Bond strength acting at the rebar surface is the interaction mechanism that enables

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the force transfer between rebar and the surrounding concrete. Hence it maintains the composite action in RC structures. When composite action is disrupted, load carrying capacity is also effected (Rodriguez et al. 1994, Coronelli 2002, Huang and Chen 2013). This in turn changes the overall behavior of the RC structures. Hence for the satisfactory performance of the RC structures adequate bond between rebar and surrounding concrete is essential.

Many experimental investigations have been undertaken during the last two decades regarding bond behavior of corroded reinforcement (Law et al. 2011). In general, these experimental investigations suggest that at low level of corrosion (<1%) bond strength increases, and with further increase in corrosion bond strength decreases significantly. Loss up to 80-90% of the initial bond strength has also been observed for only about 5-7% of the corrosion level in unconfined concrete (Rodriguez et al. 1994, Zhao et al. 2013, Banba et al. 2014). Similar results were observed from analytical and numerical studies carried out by Coronelli (2002), Lundgren (2002), Wang and Liu (2004) and Bhargava et al. (2007b). Few experimental investigations have also been carried out in order to evaluate the relationship between surface crack width and bond behavior of corroded deformed rebar (Rodriguez et al. 1994, Almusallam et al. 1996, Fischer et al. 2010, Law et al. 2011, Li and Yuan 2013, Banba et al. 2014). Some empirical relations have also been proposed to describe the influence of crack width on the bond strength of the corroded plain rebar based on the experimental results (Cairns et al. 2006). The summary of the published results on the bond strength behavior of the corroded deformed rebar with respect to cover surface crack width obtained from various reference literatures is presented in Fig. 1.

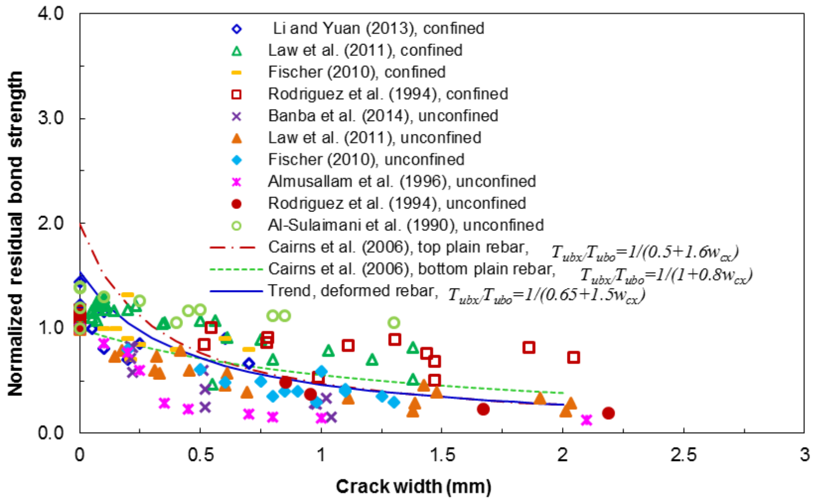

Fig. 1 Relation between normalized residual bond strength and corrosion induced cover surface crack width

The results in Fig. 1 are obtained by plotting normalized residual bond strength ratio of ultimate bond strength of the corroded element (T_{ubx}) to non-corroded element (T_{ubo}) versus cover surface crack width (w_{cx}), together with the results obtained from the empirical equations proposed by Cairns et al. (2006). Significant scatter can be observed from these experimental investigations. Therefore a trend line has been plotted, which indicates that with increase in crack width at the cover surface residual bond strength is considerably decreased. Despite the scatter and inconsistencies between the quantitative reductions in the residual bond strength, they clearly indicate that residual bond strength of the corroded rebar is significantly decreased with an increase in cover surface crack width.

Research on the prediction of the life cycle performance of corroded RC structures associated with bond strength degradation is very limited. Stochastic modelling for bond strength deterioration has significant potential for assessing the condition and life cycle performance of the RC structures. Therefore, this paper presents a time-dependent reliability analysis of corrosion affected RC structures associated with the bond strength degradation together with the comprehensive approach of gamma-process for deterioration modelling. In reliability analysis, structural failure is considered when its deterioration reaches the predefined allowable limit. The applicability of the proposed methodology is then presented with a numerical example.

2. REINFORCEMET CORROSION

Steel rebar embedded in the concrete is normally protected by a passive layer created by the high alkalinity of the concrete. This protective layer can be broken down due carbonation or the chloride ingress from the environment. Once the passive layer is broken down, corrosion initiates. The expansive layer of the corrosion product initiates cracking at the steel concrete interface and these cracks propagate toward the cover surface of the concrete cover (Pantazopoulou and Papoulia 2001). As a result, the bond strength between the steel reinforcement and the surrounding concrete starts decreasing (Law et al. 2011)

This reduction in bond strength together with the decreasing rebar size can lead to significant reduction in residual load carrying capacity and stiffness of the RC structures, after which structural collapse is most like to occur (Torres-Acosta et al. 2007, Nepal and Chen 2014). Hence, it is clear that crack propagation and reduction in residual strength can significantly affects the overall performance and shortens the service life of the RC structures suffering from reinforcement corrosion. For the time-dependent reliability analysis of these structures, quantification of these damages is essential. In this regard, the analytical investigations of these damages caused by reinforcement corrosion are now presented.

2.1 Loss of cross-sectional area of the rebar

The reduced diameter of the rebar   from its initial state   is generally evaluated in terms of attack penetration   (pitting attack or homogeneous corrosion), expressed as

$$D_{bx} = D_b - \alpha_p x \tag{1}$$

when $4 < \alpha_p < 8$ and homogeneous corrosion at later stage when $\alpha_p = 2$ (Vidal et al. 2004). According to the study carried out by Zhang et al. (2010), in long-term natural corrosion process, at the beginning although corrosion appears as the localized but in later stage it appears as uniform corrosion. Hence in this paper corrosion has been considered as uniform. Therefore, radial displacement at the rebar surface (u_{bx}) caused by expansive corrosion product is uniform and axis symmetric, expressed here as

$$u_{bx} = \frac{\Delta V}{\pi D_b} = \frac{1}{4}(\gamma_{vol} - 1) D_b X_p \tag{2}$$

where $\Delta V = V_r - \Delta V_s$ is the volume increase per unit length of the rebar in which V_r is the volume of the rust product formed given by $V_r = \gamma_{vol} A_b X_p$ and ΔV_s is the volume of loss of the corroded rebar, this can be obtained once V_r is known . γ_{vol} is the volume ratio of the corrosion product formed to its parent metal, this generally lies between1.8 to 6.4 (Pantazopoulou and Papoulia 2001, Lundgren 2002); A_b is the original cross-sectional area of the rebar and X_p is the corrosion level, defined as the ratio of the mass loss of corroded rebar to its original mass.

2.2 Cracking in concrete cover

Cracking in the concrete cover generally occurs when the hoop stress on the concrete surface reaches the tensile strength of the concrete. The concrete cracking process due to reinforcement corrosion has been investigated analytically by adopting the anisotropy of cracked concrete and thick walled cylinder model previously utilized by Chen and Xiao (2012) together with the concept of equivalent crack as utilized by Chen and Alani (2013). The schematic representation of thick walled cylinder model used in this paper is shown in Fig. 2 , in which the concrete surrounding the reinforcing rebar is considered as thick walled cylinder with wall thickness equal to clear cover depth (C).

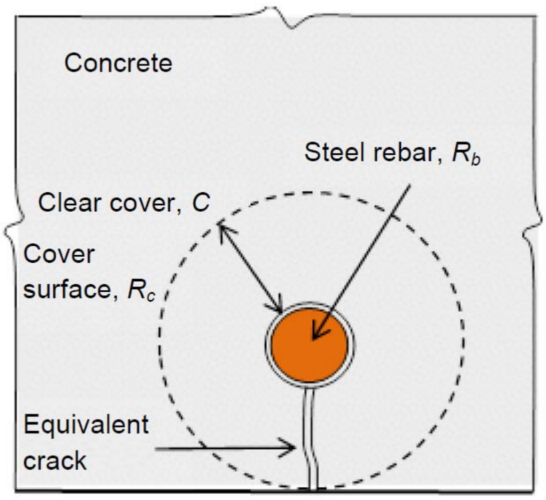

Fig. 2 Idealization of cover concrete as thick-walled cylinder model for predicting concrete crack development and bond strength evolution

The cracking in the concrete cover has been considered as cohesive in nature and residual tensile stress in the cracked concrete has been obtained by adopting bilinear stress softening law of cracked concrete as described in CEB-FIP (1990). In the cohesive crack model, the stress transferred through the cohesive cracks is assumed to be a function of crack opening (softening curve), expressed as

$$\sigma_w = \left(f_t a - bW \right) \tag{3}$$

where σ_w is the tensile stress acting across cohesive cracks; f_t is the maximum tensile strength of concrete at onset of cracking; W is the normalized crack width defined as $W = f_t w(r) / G_f$ in which G_f is the fracture energy of the concrete; $w(r)$ is the actual crack width at any point r between R_b and R_c as shown in Fig. 2.

Coefficients a and b are the bilinear coefficients, depending on the pre-critical stage ($0 \le W \le W_{cr}$) and post-critical stage ($W_{cr} \le W \le W_u$) of crack width, as

$$a = 1; b = \frac{1-\alpha_{bi}}{W_{cr}} \quad for\ pre-critical\ stage \tag{4a}$$

$$a = \frac{\alpha_{bi} W_u}{W_u - W_{cr}}; b = \frac{\alpha_{bi}}{W_u - W_{cr}} \quad for\ post-critical\ stage \tag{4b}$$

in which α_{bi} is coefficient of bilinear softening curve; W_{cr} is normalized critical crack width and W_u is ultimate cohesive crack width. The critical and ultimate cohesive crack widths can be determined from experiments for concrete. In the CEB-FIP(1990), the coefficient α_{bi} is given as α_{bi} = 0.15; W_{cr} and W_u can be evaluated from concrete strength, fracture energy and maximum aggregate size.

From the anisotropic property and the bilinear softening law of the cracked concrete, normalized crack width at the rebar surface of thick walled cylinder (R_b) can be expressed as

$$W_{bx} = \frac{1}{b(l_o - R_b)} \left(\frac{E}{f_t} u_{bx} - aR_b \right) \tag{5}$$

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 thick walled cylinder model and $l_{ch} = EG_f / f_t^{\,2}$ is the characteristic length; $E = E_c / (1+\theta_c)$ is the effective modulus of elasticity of the concrete in which E_c is the modulus of elasticity of concrete and θ_c is the creep coefficient; and u_{bx} is the radial displacement at the rebar surface as described in Eq. (2). Once the cracks initiate at the bond interface, they propagates towards the cover surface (R_c). The corrosion level at the time to crack on the cover surface ($X_p{}^c$) can be obtained from

$$X_p{}^c = \frac{2}{(\gamma_{vol}-1)}\frac{f_t}{E}\left\{1+(1+\vartheta)\frac{2R_c}{D_b}(l_o{}^{cr}-R_b)(l_o{}^{cr}-R_c)\delta^{cr}(R_c,R_b)\right\} \tag{6}$$

where ϑ is the Poisson's ratio of the concrete and $\delta(R_c,R_b)$ is crack factor denoted by $\delta^{cr}(R_c,R_b)$ and $\delta^{u}(R_c,R_b)$ in pre-critcal stage and post critical stage of the cracking, respectively, given by

$$\delta^{cr}(R_c,R_b) = \frac{R_c-R_b}{l_o{}^{cr}(l_o{}^{cr}-R_c)(l_o{}^{cr}-R_b)} + \frac{1}{(l_o{}^{cr})^2}\ln\frac{R_c}{R_b}\frac{\left|l_o{}^{cr}-R_b\right|}{\left|l_o{}^{cr}-R_c\right|} \tag{7a}$$

$$\delta^{u}(R_c,R_b) = \frac{R_c-R_b}{l_o{}^{u}(l_o{}^{u}-R_c)(l_o{}^{u}-R_b)} + \frac{1}{(l_o{}^{u})^2}\ln\frac{R_c}{R_b}\frac{\left|l_o{}^{u}-R_b\right|}{\left|l_o{}^{u}-R_c\right|} \tag{7a}$$

in which $l_o{}^{cr}$ and $l_o{}^{u}$ are the material constants for pre-critical stage and post-critical stage cracking of the concrete cover. Ignoring the Poisson's effect associated with the hoop strain of the completely cracked concrete, the normalized crack width on the concrete cover surface W_{cx} can be expressed as

$$W_{cx} = \frac{W_{bx}-\frac{a}{b}\left\{R_c(l_o-R_c).\delta(R_c,R_b)\right\}}{1-R_c(l_o-R_c).\delta(R_c,R_b)} \tag{8}$$

In cohesive crack model the process of concrete cracking continues until they reach their ultimate cohesive value at the cover surface. The corrosion level at which the equivalent crack at the cover surface reaches its ultimate cohesive value can be obtained from

$$X_p{}^u = \frac{2}{\pi(\gamma_{vol}-1)}\frac{f_t}{E}\frac{n_c l_{ch}}{D_b}W_u \tag{9}$$

At this stage due to the widening of the crack opening, the residual strength and the corrosion stress acting at the rebar surface become negligible.

2.3 Bond strength deterioration

Corrosion in the rebar affects the bond properties acting at the steel-concrete interface by changing the shape and angle of the ribs of deformed rebar. It also influences the mechanical interlocking and confinement between rebar and the surrounding concrete by reducing adhesion and frictional force caused by the accumulation of corrosion products and cracking in the concrete cover. Therefore corrosion in reinforcement threatens all these factors required for good bonding condition of the RC structures. Considering these effects, deterioration of ultimate bond strength T_{ubx} of the corroded deformed rebar can be evaluated from contributions of three stresses acting at the bond interface, i.e. adhesion stress T_{adx}, confinement stress T_{cnfx} and corrosion stress T_{corrx} (Coronelli (2002).

$$T_{ubx} = T_{adx} + T_{cnfx} + T_{corrx} \tag{10}$$

The adhesion stress acting between rebar and concrete is given by

$$T_{adx} = \frac{n_{st} A_{rx} f_{cohx} \left\{ \cot \delta_o + \tan(\delta_o + \varphi) \right\}}{\pi D_{bx} S_r} \tag{11}$$

where n_{st} is the number of stirrups provided; $A_{rx} = \pi D_{bx} h_{rx}$ is the reduced rib area in plane at right angle to rebar axis and $h_{rx} = 0.07 D_{bx}$ is the reduced rib height of the rebar due to corrosion; $f_{cohx} = 2 - 10(x - x_c)$ is the adhesion strength coefficient in which x_c is the corrosion depth corresponding to the through cracking of the concrete cover, and is obtained from $X_p{}^c$; $\tan(\delta_o + \varphi)$ can be estimated from 1.57– 0.785 x (Coronelli and Gambarova 2000) in which δ_o is the orientation of the rib usually taken as 45° and φ is the angle of friction between steel and concrete; and $S_r = 0.6 D_b$ is the rib spacing (Wang and Liu 2004).

The confinement stress is given by

$$T_{cnfx} = K_{cnfx} P_{cnfx} \tag{12}$$

where K_{cnfx} is the coefficient of confinement stress evaluated from $K_{cnfx} = n_{st} C_r \tan(\delta_o + \varphi)/\pi$ in which C_r is the shape factor constant taken as 0.8 for crescent shape rebar. In this study condition of unconfined concrete has been considered. Therefore, P_{cnfx} is the confinement stress only provided by the cracked concrete surrounding the reinforcement. By adopting the properties of the thick walled cylinder model with cohesive cracks as described in Fig. 2, Nepal et al. (2013) has modified the expression of confinement stress given by Giuriani et al. (1991), expressed here as

$$P_{cnfx} = \frac{C}{R_{bx}} \times f_t \frac{D_a (w_u - w_{bx})}{w_u (D_a + k_c w_{bx})} \tag{13}$$

where $w_{bx} = G_f W_{bx} / f_t$ is actual crack width at the rebar surface corresponding to corrosion level X_p and k_c is the constant taken as 167(Giuriani et al. 1991).

The bond strength contributed by the corrosion pressure is given by

$$T_{corrx} = \mu_x P_{corrx} \tag{14}$$

in which μ_x is the coefficient of the friction between the corroded rebar and cracked concrete defined as 0.37-0.26 ($x - x_c$) and P_{corrx} is the corrosion pressure or the radial pressure acting at the bond interface due to the accumulation of the corrosion product at the rebar surface. The corrosion pressure acting at bond interface can be evaluated from (Chen and Xiao 2012)

$$P_{corrx} = \frac{f_t}{1-\vartheta^2}\left\{\left(1+\vartheta\sqrt{\beta_{bx}}\right)\left(a - bW_{bx}\right) + \frac{\xi_{corr}}{(l_o - R_b)}\frac{b}{R_b} + \vartheta\sqrt{\beta_{bx}}\, b l_o \frac{W_{bx}}{R_b}\right\} \tag{15}$$

where ξ_{corr} is the corrosion factor given by $\xi_{corr} = -W_{bx} / \delta(R_c, R_b)$ before through cracking and $\xi_{corr} = \left(W_{cx} - W_{bx}\right) / \delta(R_c, R_b)$ after through cracking of the concrete cover and β_{bx} is the stiffness reduction factor associated with the cracked concrete, given by

$$\beta_{bx} = \frac{1}{1 + \left\{\dfrac{b l_o W_{bx}}{\left(a - bW_{bx}\right) R_b}\right\}} \tag{16}$$

3. STOCHASTIC DETERIORATION MODELLING

Deterioration caused by reinforcement corrosion and its structural response is a complex phenomenon with high possibilities of uncertainties. The uncertainties associated with the deterioration and the corresponding structural response can be dealt with the stochastic process. Hence in this study, probability of failure of the corroded RC structure is evaluated by considering the bond strength deterioration as the stochastic process. From Fig.1 it is clear that after cracking of the concrete at the cover surface, the bond strength deterioration due to reinforcement corrosion is continuous and non-negative. Therefore from the definition of gamma process (Van Noortwijk and Frangopol 2004, Chen and Alani 2012), the gamma process is suitable for the stochastic modelling of bond strength deterioration in corrosion affected RC structures during their life cycle. In gamma process deterioration model, the cumulative bond strength deterioration (J_b) is a random quantity, and has the gamma distribution with the shape parameter $\eta(w_{cx}) \succ 0$ and scale parameter $\lambda \succ 0$. The probability density function of bond strength deterioration (J_b) at crack width $w_{cx} \succ 0$ can be expressed as

$$f_{J_b(w_{cx})}\left(J_b\right) = Ga\left(J_b \middle| \eta\ w_{cx}), \lambda\right) = \begin{cases} \dfrac{\lambda^{\eta(w_{cx})}}{\Gamma\left(\eta(w_{cx})\right)} J_b^{\eta(w_{cx})-1} e^{-\lambda J_b} & \text{for } J_b \geq \\ 0, & \text{elsewhere} \end{cases} \tag{17}$$

where $\Gamma\left(\eta(w_{cx})\right) = \int_0^\infty \nu^{\eta(w_{cx})-1} e^{-\nu} d\nu$ is the gamma function for $\eta(w_{cx}) \succ 0$. 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). Assuming J_b as the average bond strength deterioration rate associated with crack width (w_{cx}), the shape function ($\eta(w_{cx})$) can be obtained from $\eta(w_{cx}) = \lambda J_b(w_{cx})$.

Assuming J_L as the maximum allowable limit of the bond strength deterioration, from the definition of probability of failure and by integrating probability density function given in Eq. (17), the lifetime distribution of failure associated with bond strength deterioration is given by

$$P_f = Pr[J_b(w_{cx}) \geq J_L] = \int_{J_b = J_L}^{\infty} f_{J_b(w_{cx})}\left(J_b\right) dJ_b = \frac{\Gamma\left(\eta(w_{cx}), J_L \lambda\right)}{\Gamma\left(\eta(w_{cx})\right)} \tag{18}$$

where $\Gamma(\eta, z) = \int_{\nu = z}^{\infty} \nu^{\eta-1} e^{-\nu} d\nu$ is the incomplete gamma function for $z \geq 0$ and $\eta \succ 0$.

4. NUMERICAL EXAMPLE

In this section the methodology mentioned in the preceding section is applied to a numerical example for a simply supported RC beam of span 5 m with minimum service life of 50 years designed to resist aggressive environment as defined by Eurocode 2. The cross-sectional width and effective depth of beam are b = 300 mm and d = 560 mm, respectively. Four rebar of a diameter 20 mm (D_b = 20 mm) are provided as the tension rebar. Two bars of a diameter 16 mm (D_{sc} = 16 mm) are provided as the compression rebar with clear cover thickness of 40 mm (C = 40 mm).

The compressive strength of concrete f_{ck} = 40 MPa and the yield strength of original reinforcing steel f_{yk} = 460 MPa with modulus of elasticity (E_{st}) = 200 GPa are adopted in this study. Material properties required for this analytical model are assumed as total crack number n_c = 4, Poisson's ratio ϑ = 0.18, creep coefficient θ_c = 2.0, mean annual current density i_{corr} = 1 μA/cm^2, fracture energy G_f = 200 N/m, density of steel ρ_s = 7850kg/m3, volume ratio of corrosion product γ_{vol} = 2.0, and the corresponding molecular weight ratio γ_{mol} = 0.725. The critical and ultimate cohesive crack width have been obtained from CEB-FIP (1990) for nominal maximum aggregate size D_a = 20 mm.

Other parameters such as f_t and E_c are evaluated from Eurocode 2.

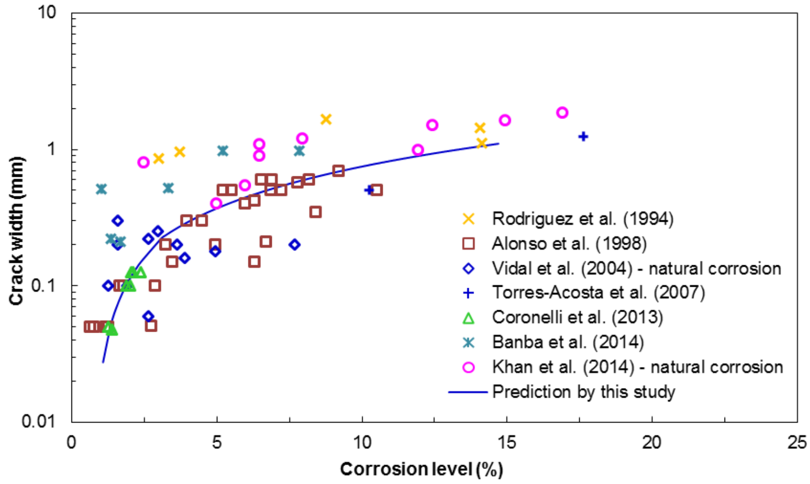

Fig. 3 Analytical prediction of cover surface crack width versus corrosion level, compared with experimental test results available from various sources

The results in Fig. 3 show the analytically predicted equivalent cover surface crack width (w_{cx}) as a function of corrosion level (X_p) in percentage. The predicted results are then compared with experimental investigations obtained from accelerated or natural corrosion tests in concrete (Rodriguez et al. 1994, Alonso et al. 1998, Vidal et al. 2004, Torres-Acosta et al. 2007, Coronelli et al. 2013, Banba et al. 2014 and Khan et al. 2014). It can be seen from Fig. 3 that the predicted crack width increases as reinforcement corrosion level increases, agreeing well with the referred experimental results.

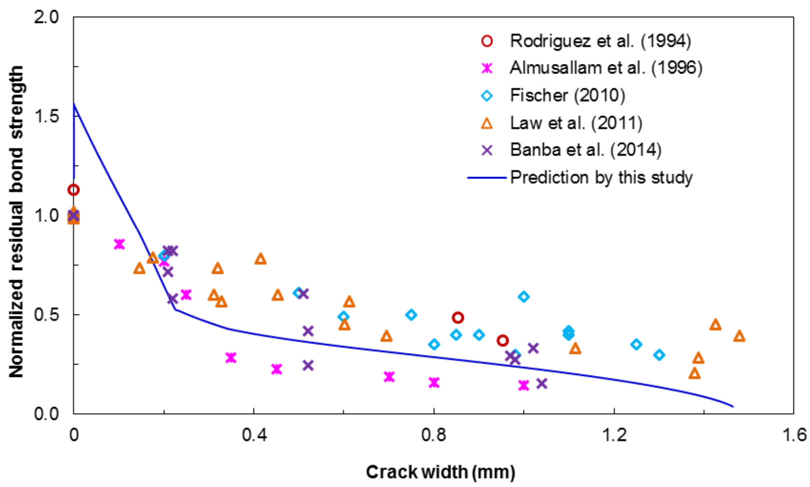

Fig. 4 Analytical prediction of normalized residual bond strength versus cover surface crack width for unconfined concrete, compared with experimental test results available from various sources

The results of normalized residual bond strength (T_{ubx}/T_{ubo}) versus equivalent cover surface crack width for unconfined concrete predicted by the present analytical study are plotted in Fig. 4. Then they are compared with the published experiment data obtained from various references (Rodriguez et al. 1994, Almusallam et al. 1996, Fischer et al. 2010, Law et al. 2011, and Banba et al. 2014). Here again, the trend of bond strength deterioration with increase in surface crack width predicted by the present study is in good agreement with the experimental investigation of the reference literatures. The residual bond strength of unconfined concrete reduces continuously with the increase in crack width and finally becomes negligible when crack width is about 1.5mm (ultimate cohesive value). This is due to the absence of transverse reinforcement (stirrups) in unconfined concrete.

The deterioration of structural performance in terms of cover surface cracking of the concrete cover is modelled as gamma process. At first, surface crack width (w_{cx}) is considered as an indicator of performance deterioration for the serviceability of the RC structure and adopted to replace J_b in Eq. (17). The lifetime distribution of probability of failure (P_f) of the corroded beam is obtained from Eq. (18) for different acceptable crack width limits, w_L = 0.3, 0.4 and 0.5 mm, respectively. The results are then presented in Fig. 5 as a function of corrosion level. As expected, the probability of failure associated with cracking of the concrete cover depends on the given acceptable crack width limit, with a higher probability of failure for a lower acceptable crack width limit. The probability of failure increases steadily with time and reaches approximately 50% when corrosion level is approximately between 5% and 8%.

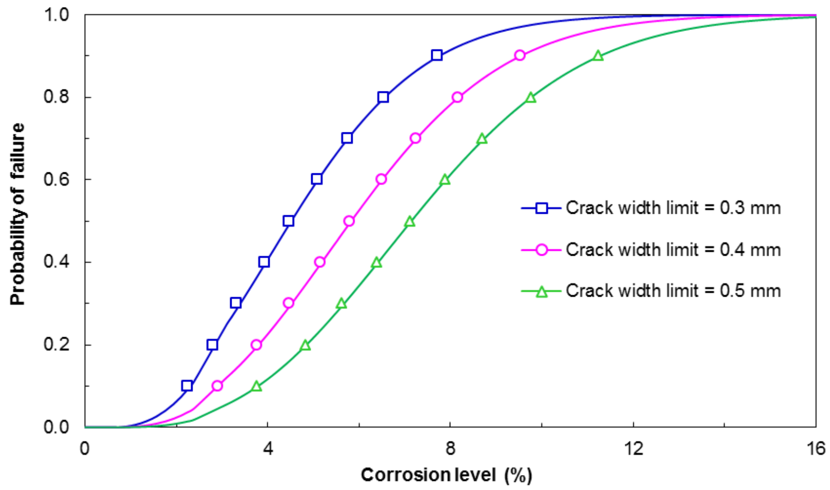

Fig. 5 Probability of failure of corroded RC structure versus corrosion level for various acceptable crack width limits

The results for probability of failure for unconfined concrete have been presented in Fig. 6. Here the deterioration of the structural performance in terms of structural capacity has been indicated by the bond strength deterioration. As shown in Fig. 4, when surface crack width is about 0.3mm, the residual bond strength of the unconfined

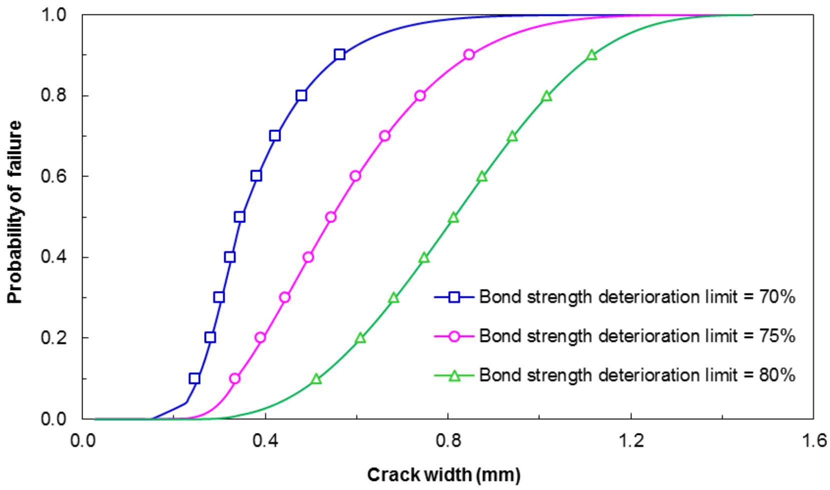

Fig. 6 Probability of failure of unconfined concrete versus surface crack width for various allowable bond strength deterioration limits

concrete has lost approximately 70% of the original strength, at the onset of cover surface cracking. Therefore to calculate probability of failure, maximum allowable limit of bond strength deterioration is considered as: J_L =70%, 75% and 80%, respectively. The probability of failure associated with the bond deterioration for different allowable limit increases steadily with increase in surface crack width. Here again, as anticipated unconfined concrete shows the highest probability of failure in lowest allowable limit of deterioration and lowest probability of failure in the highest allowable limit.

5. CONCLUSIONS

This paper presents a new approach for evaluating the life cycle performance of corrosion affected RC structures. At first, the bond strength deterioration caused by corroded reinforcement and crack growth in concrete cover is evaluated by analytical investigations. Then the analytical results are validated by experimental data available. By using the stochastic model, the probability of structural failure associated with the surface crack width and bond strength deterioration over the lifecycle of corrosion affected RC structures is evaluated. The application of the proposed approach is illustrated with the numerical example.

On the basis of the results obtained from the numerical example, following conclusions are drawn: a) The proposed approach is capable of evaluating the crack growth and residual bond strength deterioration of the corroded RC structures; b) Bond strength decreases with increase in cover surface crack width; c) The probability of failure of corrosion affected RC structures during their life cycle depends on the predefined allowable limit of their deterioration. Thus, the proposed approach is capable of assessing the life cycle performance of concrete structures affected by reinforcement corrosion.

REFERENCES

Almusallam, A.A., Al- Ghantani, A.S., Aziz, A.R. and Rasheeduzzafar (1996), "Effect of reinforcement corrosion on bond strength." *Construction and Building Materials*, Vol. **10**(2), 123-129.

Alonso, C., Andrade, C., Rodriguez, J., and Diez, J. M. (1998), "Factors controlling cracking of concrete affected by reinforcement corrosion." *Materials and Structures*, Vol. **31**, 435-441.

Banba, S., Abe,T., Nagaoka, K. and Murakami, Y. (2014), "Evaluation method for bond-splitting behavior of reinforced concrete with corrosion based on confinement stress of concrete against corrosion expansion." *Journal of Advanced Concrete Technology*, Vol.**12**(1), 7-23.

Bhargava, K., Ghosh, A. K., Mori, Y. and Ramanujam, S. (2007a), "Ultimate flexural and shear capacity of concrete beams with corroded reinforcement." *Structural Engineering and Mechanics*, Vol. **27**(3), 347–363.

Bhargava, K., Ghosh, A. K., Mori, Y. and Ramanujam, S. (2007b), "Corrosion-induced bond strength degradation in reinforced concrete- analytical and empirical models." *Nuclear Engineering and Design*, Vol. **237**(11), 1140-1157.

Cairns J., Du, Y. and Law, D. (2006), "Residual bond strength of corroded plain round bars." *Magazine of Concrete Research*, Vol. **58**(4), 221- 231.

CEB-FIP (1990), *CEB-FIP Model Code 1990*, Comité Euro-International du Béton (CEB), Thomas Telford, London, UK.

Chen, H.P. and Alani, A. M. (2012). "Reliability and optimised maintenance for sea defences." *Proceedings of the ICE - Maritime Engineering*, Vol. **165**(2), 51 –64.

Chen, H.P. and Alani, A. M. (2013). "Optimized maintenance strategy for concrete structures affected by cracking due to reinforcement corrosion." *ACI Structural Journal*, **Vol**. **110**(2), 229-238.

Chen, H.P. and Xiao, N. (2012), "Analytical solutions for corrosion-induced cohesive concrete cracking." *Journal of Applied Mathematics* , Article ID 769132.

Coronelli, D. (2002), "Corrosion cracking and bond strength modelling for corroded bars in reinforced concrete." *ACI Structural Journal* , Vol. **99**(3), 267–276.

Coronelli, D. and Gambarova, G. (2000), "A mechanical model for bond strength of corroded reinforcement in concrete", *Proceedings of the 14th Engineering Mechanics Conference*, Austin, Texas.

Coronelli, D., Hanjari, K., and Lundgren, K. (2013), "Severely Corroded RC with Cover Cracking." *Journal of Structural Engineering, ASCE*, Vol. **139**(2), 221–232.

corrosion in reinforced concrete." *Cement and Concrete Research*, Vol. **34**(1), 165-174.

Eurocode 2 (2004), *Design of Concrete Structure*, European comitee for standardization, Brussels, Belgium.

Fischer C. (2010). "Experimental investigations on the effect of corrosion on bond of deformed bars", *Proceedings of the 8th fib PhD Symposium*, Kgs, Lyngby, Denmark.

Giuriani, E., Plizzari, G. and Schumm, C. (1991). "Role of stirrups and residual tensile strength of cracked concrete on bond." *Journal of Structural Engineering, ASCE* , Vol. **117**(1), 1-18.

Huang T.L. and Chen H.P. (2013). “Symptom-based reliability analysis and remaining service life prediction of deteriorating RC structures.” *Key Engineering Materials*, Vol. **569**, 151-158.

Khan, I., Francois, R. and Castel, A. (2014), “Prediction of reinforcement corrosion using corrosion induced cracks width in corroded reinforced concrete beams.” *Cement and Concrete Research*, Vol. **56**, 84-96.

Law, D. W., Tang, D., Molyneaux, T. K. C. and Gravina, R. (2011), “Impact of crack width on bond: confined and unconfined rebar.” *Materials and Structures*, Vol. **44**(7), 1287-1296.

Li, F. and Yuan, Y. (2013), “Effects of corrosion on bond behavior between steel strand and concrete.” *Construction and Building Materials*, Vol. **38**, 413–422.

Lundgren, K. (2002), “Modelling the effect of corrosion on bond in reinforced concrete.” *Magazine of Concrete Research*, Vol. **54**(3), 165-173.

Nepal, J., Chen, H.P. and Alani, A.M. (2013), “Analytical modelling of bond strength degradation due to reinforcement corrosion.” *Key Engineering Materials* , Vol. **569**, 1060-1067.

Nepal, J. and Chen, H.P. (2014), “Evaluation of structural behavior of corrosion damaged reinforced concrete bridges.” Proceedings of 15th European Bridge Conference & Exhibition - Structural Faults and Repair 2014, London, UK.

Pantazopoulou, S. and Papoulia, K. D. (2001), “Modeling cover-cracking due to reinforcement corrosion in RC structures.” *Journal of Engineering Mechanics, ASCE,* Vol. **127**(4), 342-351.

Rodriguez, J., Ortega, L. M., and Casal, J. (1994), “Corrosion of reinforcing bars and service life of reinforced concrete structures: corrosion and bond deterioration”, *Proceedings of International Conference on Concrete Across Borders*, Odense, Denmark.

Shetty, A., Venkataramana, K. and Narayan K.S. B. (2014), “ Flexural bond strength behaviour in OPC concrete of NBS beam for various corrosion levels.” *Structural Engineering and Mechanics*, Vol. **49**(1),81-93.

Torres-Acosta, A.A., Navarro-Gutierreza, S. and Teran-Guillen J. (2007), “Residual flexure capacity of corroded reinforced concrete beams.” *Engineering Structures*, Vol. **29**(6), 1145-1152.

Van Noortwijk, J.M. (2009), “A survey of the application of gamma processes in maintenance.” *Reliability Engineering & System Safety*, Vol. **94**(1), 2–21.

Van Noortwijk, J.M. and Frangopol, D.M. (2004), “Two probabilistic life-cycle maintenance models for deteriorating civil infrastructures.” *Probabilistic Engineering Mechanics*, Vol. **19**(4), 345–359.

Vidal, T., Castel, A., and Francois, R. (2004), “Analyzing crack width to predict

Wang, X. H. and Liu, X.L. (2004). “Modelling effects of corrosion on cover cracking and bond in reinforced concrete.” *Magazine of Concrete Research*, Vol. **56**(4), 191-199.

Zhang, R., Castel, A. and Francois, R. (2010), “Concrete cover cracking with reinforcement corrosion of RC beam during chloride-induced corrosion process.” *Cement and Concrete Research*, Vol. **40**(3), 415-425.

Zhao, Y., Lin, H., Wu, K. and Jin, W. (2013), “Bond behaviour of normal/recycled concrete and corroded steel bars.” *Construction and Building Materials*, Vol. **48**, 348–359.