MODELLING RESIDUAL FLEXURAL STRENGTH OF CORRODED REINFORCED CONCRETE BEAMS

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ABSTRACT

This paper presents a new analytical method for evaluating concrete crack development, estimating rebar bond strength degradation and predicting residual flexural strength of concrete beams affected by reinforcement corrosion. First, cracking development in cover concrete due to reinforcement corrosion is investigated by using the rebar-concrete model where realistic concrete properties such as bilinear tension softening law for the cracked concrete are considered. Then, the bond strength evolution of the corroded rebar is evaluated by considering the contributions from adhesion, concrete confinement and corrosion pressure acting at the bond interface. The effects of cover

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concrete cracking on the corroded rebar bond strength are analytically investigated. On the basis of the estimated concrete crack width and rebar bond strength degradation due to corrosion, the residual flexural strength of corroded concrete beams is predicted by assuming new strain compatibility at the rebar bond interface. Finally, the results obtained from the proposed methods are examined by experimental and field data available from various sources. From the results, the residual flexural strength of corroded reinforced concrete beams can be largely dependent on the residual bond strength of corroded rebar, and the failure mode of the beams may be changed from rebar tensile yielding to rebar anchorage failure due to rebar bond strength degradation. The results also show that the proposed analytical approach is capable of providing accurate predictions for concrete cracking, bond strength degradation and residual flexural strength of corrosion damaged reinforced concrete beams.

Keywords: reinforcement corrosion; concrete cracking; bond strength; flexural strength; concrete beam.

1. INTRODUCTION

Reinforcement corrosion is a critical issue associated with the safety and reliability of reinforced concrete (RC) structures. The major causes of reinforcement corrosion can be either due to the concrete carbonation or the ingress of chloride ion in concrete structures exposed to aggressive environments such as deicing salt or marine environments. Because of reinforcement corrosion, the serviceability and bearing capacity of RC structures can be seriously affected. As a result, these corrosion affected RC structures may be unable to serve for the full service life as they are designed for. To ensure these structurally deficient RC structures are safe for use, tremendous costs therefore are needed for necessary maintenance and repairs [1, 2]. In Europe, about a half of its annual construction budget is spent on the refurbishment and repair of existing RC structures [3]. Along with these direct costs, there are also significant amount of indirect costs such as traffic delay cost and the cost associated with loss of lives and properties. As the number of corrosion affected RC structures increases with time, management and maintenance of these deteriorating RC structures have become greater challenge both economically and technically. Existing research shows that the performance deterioration of corrosion damaged RC structures is mainly associated with the change in the mechanical properties of both reinforcement and the surrounding concrete [4, 5]. This change in mechanical properties, together with decreasing size of the rebar and increasing crack width in the concrete cover as corrosion progresses, can lead to significant reduction in the load carrying capacity as well as the serviceability of corroded RC structures.

Many investigations have been carried out during the last two decades regarding the prediction of reinforcement corrosion initiation and concrete crack propagation [6-10]. Studies on bond strength deterioration have been widely undertaken such as in the work [11-17]. These studies indicate that bond strength may increases at low corrosion level (<1-2%), but significantly reduces when cracking appears at the concrete cover surface. Bond strength is the interaction mechanism that enables the force transfer between rebar and the surrounding concrete. Without sufficient bond strength, composite action in RC structures cannot occur. When composite action is disrupted, load carrying capacity (i.e. flexural capacity) of RC structures will be affected. Thus, research has been also carried out on evaluating the residual load carrying capacity of the corroded RC beams [18-21]. Torres-Acosta et al. [18] undertook an experimental investigation to give relationship between flexural capacity loss and rebar cross-section loss of RC beams due to reinforcement corrosion, and the experimental results showed that the flexural capacity was significantly reduced with only 10% of the ratio of average corrosion penetration to rebar radius. Azad et al. [20] proposed a correction factor which combines the effect of the bond strength loss and other factors relating to the loss of flexural strength to achieve the residual flexural strength. Mangat and Elgarf [22] examined the flexural performance of the corroded RC beams with different corrosion current densities and found that the bond strength loss has significant effects on the residual flexural strength, comparing with the rebar area loss in their experimental specimens. To predict flexural behaviour due to corrosion, the effect of rebar mass loss on the residual flexural strength of the corroded RC elements has been further investigated by EI Maaddawy et al. [5]. Their results showed that the flexural capacity decreases as reinforcement corrosion increases, reducing to approximately 80% of its original strength at the mass loss of 12%. Furthermore, experimental studies on the effect of reinforcement corrosion on bond strength have been carried out to investigate the flexural strength deterioration of corroded RC beams [23-27]. Also, limited research has been undertaken to develop analytical methods for predicting the residual flexural strength of corroded RC beams [28]. However, the influence of bond strength loss on the residual flexural strength of corroded RC beams is still not well understood, in particular their analytical relationship. Although some analytical models have been proposed for modelling cover concrete cracking due to reinforcement corrosion [8, 30, 31], there is a lack of theoretical models which can effectively evaluate the residual flexural strength of RC beams with consideration of realistic behaviour of concrete cracking and bond strength degradation. Therefore, there is a need to develop a reliable model for predicting the evolution of residual rebar bond strength and flexural strength of RC beams subject to reinforcement corrosion. The research in this paper aims to develop an accurate method for modelling concrete cover cracking, rebar bond strength degradation and residual flexural strength of corroded RC beams.

2. RESEARCH SIGNIFICANCE

The mechanism of cover concrete cracking due to reinforcement corrosion and its impact on flexural load carrying capacity of concrete beams are complex, thus there is a lack of availability of accurate analytical models for estimating concrete cracking and load carrying capacity deterioration. As a result, there is significant difficulty in accurately modelling and predicting the behaviour of corroded concrete beams when the realistic properties of cracked concrete due to reinforcement corrosion are considered, although there are some experimental results available from laboratory tests such as in experimental studies [18. 20, 22]. This paper presents an accurate analytical method for evaluating the concrete crack development and residual load carrying capacity of corroded RC beams. On the basis of the rebar-concrete model, the crack width growth in the concrete cover is determined as reinforcement corrosion progresses. From the obtained concrete cracking evolution, the bond strength degradation of the corroded rebar is estimated, and then the flexural strength deterioration of corroded RC beams is predicted. Finally, the proposed models for predicting concrete cracking and residual load carrying capacity of corroded concrete beams are examined by experimental data available.

3. REINFORCEMENT CORROSION

The performance deterioration process of corrosion affected RC structures can be schematically represented by Figure 1, and is divided here into three main phases, i.e. crack initiation phase, crack propagation phase and residual life phase. The deterioration could start in crack initiation phase although structural performance may not deteriorate significantly at this stage. The deterioration rate then increases gradually in crack propagation phase and accelerates during the residual life phase until the structural resistance reaches to the ultimate limit, and finally the structure collapses. Existing experimental studies [9, 18, 20, 29] show that reinforcement corrosion affects the residual strength RC structures in many ways, including: a) loss of rebar cross-section, b) reduction in yield strength of rebar, c) cracking in concrete cover, and d) deterioration of bond strength acting at the steel-concrete interface. In order to correctly evaluate the flexural strength of RC structures affected by reinforcement corrosion, these factors need to be considered in analysis.

In general, loss in rebar cross section is represented by the mass loss or the cross sectional area loss of the rebar. Therefore, the reduced diameter of the rebar D_{bx} from its original dimension D_b can be estimated in terms of attack penetration x (pitting or homogeneous corrosion), as defined in Vidal et al. [6], expressed here as

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

where α_p is an attack penetration factor indicating localised corrosion at the earlier stage when $4 < \alpha_p < 8$ and homogeneous corrosion at later stage when α_p = 2. Hence, the corresponding corrosion level X_p is defined as the ratio of the mass loss of the corroded rebar ΔM_s to the original mass of the rebar M_o, namely

$$X_p = \frac{\Delta M_s}{M_o} = \frac{\Delta A_b}{A_b} = 1 - \frac{{D_{bx}}^2}{{D_b}^2} \tag{2}$$

where ΔA_b is the loss of cross-sectional area of the corroded rebar and A_b is the cross-sectional area of the original rebar. The mass of the rust products formed during corrosion process M_r can be obtained from $M_r = \Delta M_s / \gamma_{mol}$ and the corresponding density of the rust products can be determined from $\rho_r = \rho_s / \gamma_{mol}\gamma_{vol}$, in which ρ_s is the density of steel, γ_{mol} is the corresponding molecular weight ratio ranging from 0.78 to 0.35, and γ_{vol} is the volume ratio of the corrosion products to its parent metal ranging from 1.7 to 6.5 and [30, 31]. From Eq. (2), the volume of the rust products V_r is calculated from $V_r = \gamma_{vol} A_b X_p$. The corresponding volume increase per unit length of the rebar is then given by $\Delta V = V_r - \Delta V_s$, where $\Delta V_s = \Delta A_b = A_b X_b$, as given in Eq. (2), is the rebar volume loss per unit length, leading to $\Delta V = (\gamma_{vol}-1) A_b X_b$. The radial displacement at the bond interface u_{bx}, generated by the increase in volume ΔV, is then expressed as

$$u_{bx} = \frac{\Delta V}{\pi D_b} = \frac{1}{4}\left(\gamma_{vol} - 1\right) D_b X_p \tag{3}$$

The prescribed displacement u_{bx} related to corrosion level X_p will be considered as the boundary condition of the boundary-value problem for analysing concrete cracking development and predicting bond strength evolution. Existing research shows that in the initial phases corrosion appears as the localised, but in later stages it appears as uniform corrosion [6, 10]. Hence reinforcement corrosion and the generated displacement at rebar surface u_{bx} are considered as uniform in this paper. This assumption for uniform displacement at the bond interface is reasonable and has been utilised in many studies such as [8, 25, 30, 32].

4. CORROSION INDUCED CONCRETE CRACKING

The concrete cracking process due to reinforcement corrosion has been investigated analytically by adopting the anisotropy of cracked concrete and the thick walled cylinder model shown in Figure 2 [7, 8, 30-32]. Due to the expansive nature of corrosion products, two types of stresses are formed in the concrete cover, namely hoop stress σ_θ and radial stress σ_r. When the hoop stress reaches the tensile strength of concrete, the radial splitting cracks appear at the bond interface ($R_b = D_b/2$) and then propagate towards the free surface of concrete cover ($R_c = C + D_b/2$), as indicated in Figure 2. In this study, the stress state of the cover concrete caused by load is not included in evaluating corrosion induced concrete cracking, although it could be taken into account in the proposed model through the superposition principle. Cracks in the concrete cover are considered as cohesive in nature and residual tensile stress in the cracked concrete is obtained by adopting bilinear stress softening law of cracked concrete as described in CEB-FIP [33] and shown in Figure 3. This softening curve gives reasonable approximations of cracked concrete in tension, expressed here as

$$\sigma_\theta = \sigma_w = f_t\left(a - bW\right) \tag{4}$$

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

$$a = a^{cr} = 1 \quad ; \quad b = b^{cr} = \frac{1-\alpha_{bi}}{W_{cr}} \text{ for pre-critical cracking stage} \tag{5a}$$

$$a = a^{u} = \frac{\alpha_{bi} W_u}{W_u - W_{cr}} \; ; \; b = b^{u} = \frac{\alpha_{bi}}{W_u - W_{cr}} \text{ for post-critical cracking stage} \tag{5b}$$

in which α_{bi} is coefficient of bilinear softening curve, given as α_{bi} = 0.15 in the CEB-FIP [33]; W_{cr} and W_u are normalised critical and ultimate crack widths, respectively, which can be determined from experiments for concrete or evaluated from concrete strength, fracture energy and maximum aggregate size D_a [33].

In an anisotropic thick walled cylinder subject to asymmetric actions, the radial strain ε_r and hoop strain ε_θ are only related to the radial displacement u at the point r of the cracked concrete cover. From the definition of hoop strain and radial strain and then by using aforementioned bilinear softening law of the cracked concrete, the radial displacement and corresponding strain can be expressed, respectively, as

$$u = \varepsilon_\theta r = \frac{f_t}{E}\left[(a - bW)r + bl_0 W\right] \tag{6a}$$

$$\varepsilon_r = \frac{du}{dr} = \frac{f_t}{E}\left[(a - bW) + b(l_0 - r)\frac{dW}{dr}\right] \tag{6b}$$

By using the governing equations for the boundary value problem of thick walled cylinder where the cracked concrete cover is modelled as axisymmetric elastic continuum, the general solution of normalised crack width is obtained [2, 8] and expressed here as

$$W = C_1 \delta(l_o, r) + C_2 \tag{7}$$

where constant coefficients C_1 and C_2 in the general solution can be determined from two boundary conditions of the boundary-value problem, depending on the phase of crack development in the concrete; $\delta(l_0, r)$ is the crack function associated with the material properties and radial distance r, given by

$$\delta\left(l_o, r\right) = \frac{1}{l_o\left(l_o - r\right)} - \frac{1}{l_o^2}\ln\frac{\left|l_o - r\right|}{r} \tag{8}$$

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 and l_{ch} is the characteristic length defined as $l_{ch} = EG_f / f_t^2$ [2, 8]; E is the effective modulus of elasticity of intact concrete defined as $E = E_c/(1+\theta_c)$ in which E_c is the modulus of elasticity of concrete and θ_c is the creep coefficient. From the relationship between stress and strain with consideration of tensile stiffness reduction, the radial stress of the cracked concrete and stiffness reduction factor β are expressed, respectively, as

$$\sigma_r = \frac{E}{1-\vartheta^2}(\varepsilon_r + \vartheta\sqrt{\beta}\varepsilon_\theta) = \frac{f_t}{1-\vartheta^2}\left[(1+\vartheta\sqrt{\beta})(a-bW) + b(l_0 - r)\frac{dW}{dr} + \vartheta\sqrt{\beta}bl_0\frac{W}{r}\right] \tag{9}$$

$$\beta = \frac{E_\theta}{E} = \frac{(a-bW)r}{(a-bW)r + bl_0 W} \tag{10}$$

Once the cracks initiate at the bond interface, they propagates towards the cover surface R_c. By considering free surface condition at the cover surface and ignoring the Poisson's effect associated with the hoop strain of the completely cracked concrete (e.g. Poisson's ratio $\vartheta \approx 0$), the normalised crack width on the concrete cover surface W_{cx} can be obtained from

$$W_{cx} = \frac{1}{1-\Delta(R_c, R_b)}[W_{bx} - \frac{a}{b}\Delta(R_c, R_b)] \tag{11}$$

where $\Delta(R_c,R_b)=R_c(l_0-R_c)\delta(R_c,R_b)$ in which $\delta(R_c,R_b)=\delta(l_o,R_c)-\delta(l_o,R_b)$ is the crack factor and parameters $\delta(l_o,R_b)$ and $\delta(l_o,R_c)$ can be obtained from Eq. (8) by considering $r=R_b$ and $r=R_c$, respectively; W_{bx} is the normalized crack width associated with radial displacement at rebar surface u_{bx}, obtained by considering $r=R_b$ in Eq. (6a) and given as

$$W_{bx}=\frac{1}{b(l_o-R_b)}\left(\frac{E}{f_t}u_{bx}-aR_b\right) \tag{12}$$

By using the two boundary conditions of the boundary value problem, i.e. the hoop stress at the concrete cover surface reaching the concrete tensile strength and normalized crack width at bond interface in Eq. (12), the corrosion level at the time to crack on the cover surface $X_p{}^c$ can be obtained from

$$X_p{}^c=\frac{2}{(\gamma_{vol}-1)}\frac{f_t}{E}\left[1+(1+\vartheta)\frac{2R_c}{D_b}\left(l_o{}^{cr}-R_c\right)\left(l_o{}^{cr}-R_b\right)\delta^{cr}\left(R_c,R_b\right)\right] \tag{13}$$

where the quantities with subscript *cr* indicates the aforementioned material constants and crack factor for pre-critical cracking stage of the concrete cover. In cohesive crack model the process of concrete cracking continues until crack width reaches the ultimate cohesive value. Therefore, the corrosion level at which the equivalent crack in the concrete cover reaches its ultimate cohesive value can be obtained from Eq. (3) and Eq. (6), as

$$X_p{}^u=\frac{2}{\pi(\gamma_{vol}-1)}\frac{f_t}{E}\frac{n_c l_{ch}}{D_b}W_u \tag{14}$$

At this stage when concrete crack width reaches their ultimate cohesive value, the radial stress acting at the rebar surface becomes negligible.

5. BOND STRENGTH DEGRADATION

Efficient and reliable force transfer between the reinforcement and the surrounding concrete is the fundamental requirement for effective performance of RC structures. Hence the evaluation of bond strength degradation is essential for the performance assessment of the corroded RC structures. Corrosion in reinforcement affects the bond properties between the rebar and the surrounding concrete. These changes are mainly associated with the reduction in adhesion and frictional force caused by the accumulation of corrosion products and cracking in the concrete cover [12, 13]. To consider these effects on the bond behavior of the corroded deformed rebar, Coronelli [13] defined the ultimate bond strength T_{ubx} related to corrosion level as the sum of three stresses acting at the bond interface, i.e. adhesion stress (T_{adx}), confinement stress (T_{cnfx}) and corrosion stress (T_{corrx}), expressed here as

$$T_{ubx} = T_{adx} + T_{cnfx} + T_{corrx} \tag{15}$$

The adhesion stress T_{adx} acting between the rebar and the surrounding concrete [13, 32] is given by

$$T_{adx} = \frac{n_{st} A_{rx} f_{cohx} \left[cot\, \delta_o + tan\left(\delta_o + \varphi \right) \right]}{\pi D_{bx} S_r} \tag{16}$$

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 in which $h_{rx} = 0.07 D_{bx}$ is the reduced rib height of the rebar due to corrosion; $S_r = 0.6 D_b$ is the rib spacing [32]; $f_{cohx} = 2 - 10\left(x - x_c \right)$ is the adhesion strength coefficient in which x_c is the corrosion depth corresponding to the thorough cracking of the concrete cover, and can be obtained once ${X_p}^c$ is known; $tan\left(\delta_o + \varphi \right)$ can be estimated from $1.57 - 0.785x$ [12] 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.

The confinement stress contributed by the surrounding cracked concrete and stirrup is given by $T_{cnfx} = k_{cnfx} P_{cnfx}$ where k_{cnfx} is the coefficient of confinement stress and P_{cnfx} is the confinement pressure [13, 34]. Coefficient of confinement stress is evaluated from $k_{cnfx} = n_{st} C_r \, tan\left(\delta_o + \varphi\right)/\pi$ in which C_r is the shape factor constant taken as 0.8 for crescent shape rebar. In unconfined concrete structures, confinement pressure P_{cnfx} is only provided by the cracked concrete $P_{cnfx,c}$. In case of confined concrete, the confinement pressure is the total contribution of cracked concrete $P_{cnfx,c}$ and the stirrups $P_{cnfx,st}$, expressed as

$$P_{cnfx,c} = \frac{2C}{D_{bx}} \times f_t \frac{D_a\left(w_u - w_{bx}\right)}{w_u\left(D_a + k_c w_{bx}\right)} \tag{17a}$$

$$P_{cnfx,st} = \frac{n_{st} A_{st}}{D_{bx} S_{st}} \times E_{st} \sqrt{\frac{a_2 w_{bx}{}^2}{\alpha_{st}{}^2 D_{st}{}^2} + \frac{a_1 w_{bx}}{\alpha_{st} D_{st}} + a_o} \tag{17b}$$

where $w_{bx} = G_f W_{bx}/f_t$ is actual crack width at the rebar surface associated with corrosion level; k_c is the constant taken as 167; D_a is the maximum aggregate size; A_{st} is the cross-section area of stirrup leg with diameter of D_{st}; S_{st} is the spacing of stirrup; E_{st} is the modulus of elasticity of steel; α_{st} is the shape factor of stirrup taken as 2; a_2, a_1 and a_o are the coefficients related to the simplified trilateral local bond-slip law of the stirrups, as given in Giuriani et al. [34]. From Eq. (17a) it is clear that the confinement stress provided by the cracked concrete depends on crack width at the rebar surface w_{bx}. Therefore, with increase of w_{bx} the confinement stress provided by the cracked concrete decreases and ultimately becomes negligible when crack width reaches its ultimate cohesive value w_u.

The ultimate bond strength contributed by the corrosion pressure is expressed as $T_{corrx} = \mu_x P_{corrx}$ in which μ_x is the coefficient of the friction between the corroded rebar and the cracked concrete defined as $0.37-0.26(x-x_c)$ [12], and P_{corrx} is the corrosion pressure or the radial stress σ_r acting at the bond interface R_b due to the accumulation of the corrosion product. The corrosion pressure associated with concrete crack width is determined by Eq. (9), where $r = R_b$.

6. FLEXURAL STRENGTH DETERIORATION

In this study, the flexural strength deterioration of RC beams affected by bond strength degradation due to reinforcement corrosion is investigated. In the intact condition without rebar corrosion, the ultimate bond strength $T_{ub,rqd}$ and the corresponding development length l_d required to prevent anchorage (bond) failure of the tensile steel rebar can be obtained from design codes such as Eurocode 2 [35], expressed here as

$$T_{ub,rqd} = \frac{f_{yd} D_b}{4 l_d} \quad , \quad l_d = \alpha_{bd} \frac{D_b f_{yd}}{4 f_{bd}} \tag{18a,b}$$

where f_{yd} is the design strength of tensile steel rebar given by f_{yk}/γ_s in which f_{yk} is the characteristic tensile strength and γ_s is the partial factor of safety of the steel rebar, taken here as 1.0 in estimating the strength of existing structures; f_{bd} is design bond strength obtained from $f_{bd} = 0.315 f_{ck}^{\ 0.67}$ for concrete strength $f_{ck} \leq 60$ MPa (8.70 ksi) and rebar diameter $D_b \leq 32$ mm (1.26 in.); and α_{bd} is the coefficient depending on many factors including the shape of anchorage, types of confinement provided by the stirrups and concrete cover.

To consider the effect of bond strength degradation on evaluating flexural strength of corroded RC beams, a typical cross section of doubly reinforced RC beam shown in Figure 4(a) is now considered. The strain and stress distributions across the beam section under initial un-corroded condition of rebar are shown in Figure 4(b) and Figure 4(c), respectively, as given by Eurocode 2. The symbols used in Figure 4 are defined as: b = width of beam; D = overall depth of the beam;

$d = D - C - D_b/2$ is effective depth of beam; $d^{'} = C + D_{sc}/2$ is the distance from centroid of the compressive steel rebar to edge of the compressive fibre; A_b = initial area of un-corroded tensile steel rebar; A_{sc} = initial area of un-corroded compressive rebar with diameter of D_{sc}; $\varepsilon_{cc} = 0.0035$ is ultimate strain of concrete; ε_{st} = strain of tensile rebar; ε_{sc} = strain of compressive rebar; Y = neutral axis depth from the edge of compressive zone; f_{st} = tensile stress acting at the centroid of tensile steel; $f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$ is the design strength of the concrete in which α_{cc} is the constant taken as 0.85 for $f_{ck} \leq 50$ MPa (7.25 ksi), f_{ck} is the characteristic compressive strength of the concrete and γ_c = partial factor of safety of the concrete taken as 1.0 here in evaluating the strength of existing structures; s is equivalent compressive zone given by $s = \lambda^{'} Y$; and η and $\lambda^{'}$ are coefficients taken as 1 and 0.8 for $f_{ck} \leq 50$ MPa (7.25 ksi), respectively.

During the process of reinforcement corrosion, when the existing ultimate bond strength of corroded rebar (T_{ubx}) is sufficient to prevent the RC beam from the bond failure ($T_{ub,rqd}$), the flexural capacity of the RC beam can be obtained by the conventional method based on compatibility condition. As the corrosion progresses, the ultimate bond strength of the corroded reinforcement T_{ubx} decreases and then becomes less than the required bond strength $T_{ub,rqd}$. In this case, due to insufficient bond strength at the bond interface, anchorage failure occurs. Hence the uniform tensile force f_{stx} generated in the corroded tensile rebar is governed by ultimate bond strength, given by

$$f_{stx} = n_b \pi D_{bx} l_d T_{ubx} \tag{19}$$

where n_b is the number of the bottom tensile rebar. In case of un-corroded perfectly bonded beam, strain compatibility at all sections exists, as given by design codes. But for the corroded beam the strain compatibility can be considered between un-bonded and perfectly bonded condition [21, 36].

Assuming the deformation of concrete at the rebar surface is mainly due to plastic deformation occurring within the plastic equivalent region (L_{eq}), new strain compatibility of corroded beam can be expressed as

$$\frac{\varepsilon_{stx}}{\varepsilon_{ccx}} = g_x \frac{d_x - Y_x}{Y_x} \quad , \quad \frac{\varepsilon_{scx}}{\varepsilon_{ccx}} = g_x \frac{Y_x - d'_x}{Y_x} \tag{20a,b}$$

where the quantities with subscript x are associated with corrosion level X_p and g_x is the interpolation factor between un-bonded and perfectly bonded conditions, expressed in [21, 36] as

$$g_x = 1 - \left(1 - \frac{T_{ubx}}{T_{ubo}}\right)\left(1 - \frac{L_{eq}}{l_d}\right) \tag{21}$$

in which plastic equivalent region $L_{eq} = 9.3Y_x$; T_{ubo} is the ultimate bond strength of un-corroded rebar and can be obtained by considering corrosion level X_p as zero in Eq. (15). Consequently strain acting at steel rebar is given by

$$\varepsilon_{stx} = \frac{f_{stx}}{A_{bx}E_{st}} \tag{22}$$

where A_{bx} is the cross-sectional area of corroded tensile rebar associated with corrosion level. The residual flexural strength can be evaluated by utilising the concept described in [37, 38], i.e. the corroded beam still follows the condition of equilibrium of resultant tensile and compressive forces acting at the beam section for different failure modes. Failure modes of flexural strain at compressive fibre and tensile fibre can be determined by satisfying the limited values of ε_{stx} , ε_{ccx} and ε_{scx} at $\varepsilon_{cc} = 0.0035$ and $\varepsilon_{st} = 0.002$, as given by Eurocode 2. In general, when tensile rebar reaches its yielding stage, compressive rebar should be close to or at its yielding stage, as designed. Here, only yielding of tensile rebar is considered. During the corrosion process, when anchorage failure occurs before yielding of both the tensile rebar and the concrete (i.e. $\varepsilon_{stx} \le 0.002$ and $\varepsilon_{ccx} \le 0.0035$), the tensile stress acting along the corroded rebar f_{stx} is governed by the bond strength and hence can be determined from Eq. (19). From equilibrium of resultant tensile and compressive forces acting at beam section, neutral axis depth Y_x is obtained from

$$Y_x = \frac{f_{stx} - f_{scx}}{\eta \lambda f_{cd} b} \tag{23}$$

where $f_{scx} = f_{ydx} A_{scx}$ is the compressive force acting at the centroid of compressive steel in which A_{scx} is the area of the compressive steel rebar; $f_{ydx} = \left(1 - 0.5X_p\right) f_{yd}$ is the residual yield strength of corroded steel rebar corresponding to corrosion level X_p [4]. By taking moment at the centroid of the tensile rebar, the residual flexural strength of the corroded RC beam is given by

$$M_{ux} = f_{ccx}\left(d_x - \frac{s_x}{2}\right) + f_{scx}\left(d_x - d'_x\right) \tag{24}$$

where $f_{ccx} = \eta \lambda f_{cd} b Y_x$ is the compressive force of the concrete. In the case when the yielding of steel occurs before the anchorage failure (i.e. $\varepsilon_{stx} > 0.002$ and $\varepsilon_{ccx} \le 0.0035$), tensile force is governed by the residual yield strength of the corroded rebar f_{ydx} and is calculated from $f_{stx} = f_{ydx} A_{bx}$.

From the equilibrium of forces, Y_x in Eq. (23) can be obtained by using the tensile force f_{stx}. Once Y_x is available the corresponding flexural strength is determined from Eq. (24). If both the tensile rebar and the concrete yield before anchorage failure (i.e. $\varepsilon_{stx} > 0.002$ and $\varepsilon_{ccx} > 0.0035$), the strain of steel rebar will be governed by the yielding of the concrete. By using $\varepsilon_{ccx} = \varepsilon_{cc} = 0.0035$, the strain of steel rebar ε_{stx} is obtained from Eq. (20a). The corresponding tensile stress f_{stx} and the neutral axis depth Y_x are then evaluated from Eq. (22) and Eq. (23), respectively. Finally, the corresponding flexural strength of the corroded RC structure M_{ux} is determined from Eq. (24).

7. MODEL EVALUATION AND APPLICATIONS

Validation of cover concrete crack growth model

In order to demonstrate the effectiveness of the proposed method, a typical corrosion affected RC beam used by Mangat and Elgarf [22] is employed in this study. In their experimental investigations, a total of 111 under-reinforced RC beam specimens, divided in nine groups (Groups 1-9), were subjected to accelerated corrosion damage and then tested under four point loading to evaluate the ultimate flexural strength. In this study, the beam specimen in Group 6 is adopted for analysis, as shown in Figure 5. The RC beam was singly reinforced with two reinforcing bars as the tensile steel with clear cover depth of 20 mm (0.79 in.) and subjected to accelerated corrosion of 3 $\mu A/cm^2$ (19.35 $\mu A/in.^2$). The reinforcing bars were 10 mm (0.39 in.) in diameter and 1100 mm (43.31 in.) long, including the anchorage length in the form of U-shaped hooks at both ends. No stirrups were provided in the beam specimens, instead shear reinforcement was provided by means of external tubular collars so as to prevent shear failure and to ensure the development of full flexural resistance and typical flexural failure in the middle-third of beam span. The yield strength of the reinforcement was 520 MPa (75.40 ksi) and the modulus of elasticity was 206 GPa (29870 ksi). The average compressive strength of the concrete cubes after 28 days was 40 MPa (5.80 ksi) and the maximum aggregate size was 10 mm (0.39 in.). During the experiments, the failure of the corroded beams was initiated by bond failure at the longitudinal reinforcement interface. Therefore, the moment of resistance of the corroded beam was controlled by the anchorage (bond) of the bars rather than the yielding of fully bonded tensile reinforcement at failure. The details of other material properties of the concrete considered for the validation of the proposed model are given in Table 1. Here, crack width in the cover concrete is represented by the equivalent crack width defined in [9, 10] as the cumulated crack width over the concrete cover. Other concrete properties such as concrete fracture energy G_F = 160 N/m (0.91 lbf/in.) and total number of crack n_c = 4 are estimated by using the measured concrete compressive strength [33]. Here, the volume ratio of the corrosion products is taken as 3.0 [30].

The results in Figure 6 show the analytically predicted equivalent cover surface crack width as a function of corrosion level in percentage. The predicted results are then compared with published experimental data (accelerated or natural corrosion) obtained from various references [6, 9, 10, 11, 18]. It can be seen from Figure 6 that the predicted crack width increases as reinforcement corrosion level increases, agreeing well with the available experimental results, in particular with the measured crack width in the condition of natural corrosion [6, 10].

Validation of reinforcing bar bond strength degradation model

The results in Figure 7 shows the results of normalized residual bond strength versus corrosion level predicted by the present analytical method, which are compared with the published experimental and field test data obtained from various references [11, 14, 16, 29, 39, 40]. The normalized bond strength associated with corrosion level is obtained by dividing the ultimate bond strength of corroded rebar by the ultimate bond strength of non-corroded rebar. Here again, the trend of bond strength evolution predicted by the present study is in good agreement with the experimental data. At low corrosion level (<1%), bond strength increases by about a half, but further increase in corrosion leads to considerable reduction of bond strength. This rapid reduction in bond strength is associated with many factors including the reduction of corrosion stress and confinement stress. It is interesting to see that bond strength deterioration of the laboratory experimental data is well consistent with that reported in the filed study of Ullasund bridge [40].

Cracking in the concrete cover is the visible sign of defects caused by reinforcement corrosion. Moreover, cracking in concrete cover is a key parameter which helps in condition monitoring of the RC structures. It is necessary to predict residual load carrying capacity such as bond strength degradation from the observable surface cracking condition. Figure 8 shows the results of normalized residual bond strength versus surface crack width for unconfined specimen, compared with the published experiment data obtained from various literatures [23-26, 29]. Here again, the predicted trend of bond strength degradation with increase in surface crack width is in good agreement with the experimental investigations of the available literatures. At the initial stage of surface cracking, the bond strength is about 50% higher than that in the non-corroded stage. The bond strength decreases considerably with further increase in surface crack width, and lost 50% of initial strength (non-corroded) stage when the surface crack width reaches about 0.2 mm (0.0079 in.). Further progress of cracking causes significant reduction in bond strength, and the bond strength of unconfined specimen becomes negligible when the crack width reaches approximately 1.5 mm (0.059 in.).

Figure 9 shows the predicted residual bond strength of confined specimen as function of surface crack width, compared with the published experimental data obtained from various sources [24, 26, 27, 29]. Despite the lower value of normalized residual bond strength in predicted results, in general the predicted trend for residual bond strength degradation of confined specimen with respect to surface crack width agrees well with the available experimental data. The lower value of normalized residual bond strength might be due to the difference in material properties, concrete geometry and the rate of corrosion density adopted in this study from those used in the experimental investigations. Similar to results for the unconfined specimen in Figure 8, the residual bond strength of confined specimen decreases with increase in surface crack width. However, in case of confined specimen, residual bond strength still exist when crack width reaches about 1.5 mm (0.059 in.) at ultimate cohesive value. This is due to the fact that in confined specimen stirrup provides some residual confining action together with the cracked concrete cover.

Validation of residual flexural strength model

The results for residual flexural strength predicted by the present method are plotted in Figure 10 as a function of the corrosion level and compared with the published experimental data. Here, the normalized residual flexural strength is calculated by dividing the flexural capacity of the corroded beam by the capacity of the non-corroded beam. The reduction in cross-sectional area of the reinforcing bar due to corrosion is considered in calculations. As observed in Figure 10, the results by the present study shows very good agreement with the experimental data of Mangat and Elgarf [22] and the data published in other experimental investigations [18, 20]. At the 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 significant decrease in bond strength, which is required to prevent the RC beam from bond failure. In addition, the residual flexural strength of the beam is calculated by ignoring the bond strength loss and by using the standard expression for the moment of resistance of under-reinforced beams given in Eurodcode 2 [35], as plotted in Figure 10. It can be seen that in the case without bond strength influence, the reduction in flexural strength follows approximately linear relation with corrosion level. The reduction in flexural strength for the case without bond strength influence is relatively low in comparison with the case with influence of bond strength loss. For instance, at a corrosion level of 20%, the residual flexure strength by using conventional method is about 80% of the original strength, whereas the corresponding flexural strength with considering influence of bond strength loss is only about 25%. This indicates that at relatively high corrosion level (>5%), bond strength reduction at the steel-concrete interface is the primary factor responsible for the deterioration of the flexural strength of the corroded beam rather than the reduction in cross sectional area of the rebar.

By comparing the deterioration process of rebar bond strength in Figure 7 and flexural strength in Figure 10 due to reinforcement corrosion, it can be observed that at corrosion level of about 5%, bond strength decreases by about 60% whereas flexural strength decreases by only 10%. When corrosion level exceeds 5%, there is significant reduction in flexural strength, which is caused by the decrease in bond strength, indicating that bond failure occurs before yielding of the steel rebar and the surrounding concrete. It is interesting to see that at about corrosion level of 15%, bond strength reduces to only 25% of its original strength while flexural capacity maintains 45% of its original capacity. This clearly shows that reinforcement corrosion has more severe effect on rebar bond strength than on the flexural strength of the corroded RC beam.

The relationship between load carrying capacity and cover surface cracking of corrosion damaged RC beams is useful for engineers to assess the condition and reliability of RC beams. Figure 11 shows the results for the effect of cover surface cracking on the structural behaviour of the corroded RC beam, e.g. rebar bond strength degradation and flexural strength deterioration. The relationship between the normalized bond or flexural strength and the equivalent cover surface crack width is then provided. Both rebar bond strength and flexural strength of the RC beam continuously decrease with the increase of crack width at the concrete cover surface. From the results, the rebar bond strength is severely reduced immediately after cover concrete is cracked, while flexural strength starts to have more reduction when crack width becomes over the typical allowable crack width limit for serviceability, e.g. 0.4mm (0.0158 in.) The results indicate that the bond strength is much more affected by cover surface cracking than the flexural strength.

8. CONCLUSIONS

In this paper a new analytical method for evaluating the residual load carrying capacity of corrosion affected RC beams is proposed. The bond strength deterioration caused by reinforcement corrosion is investigated analytically where the crack growth in concrete cover is considered. The flexural strength of corroded RC beams is evaluated by considering the bond strength degradation due to reinforcement corrosion. The rebar bond degradation and flexural strength deterioration, together with concrete crack growth, predicted by the proposed methods are then validated by experimental and field data available from various sources.

From the results for the case studies obtained from the proposed models, following conclusions can be drawn: 1) The proposed analytical models consider realistic properties of cracked concrete due to reinforcement corrosion, and these models agree well with the experimental data available and give accurate predictions for the performance deterioration of corroded RC beams; 2) Reinforcement corrosion has more impact on rebar bond strength, comparing with the flexural strength. Rebar bond strength degradation due to reinforcement corrosion may cause the change of the failure mode of RC beams to rebar anchorage failure; 3) Flexural strength of RC beams decreases considerably as reinforcement corrosion progresses due to significant reduction in residual bond strength, which indicates that the bond strength degradation of corroded rebar can be a dominant factor causing deterioration of the flexural strength; 4) The proposed models are capable of evaluating the concrete crack growth, rebar bond strength degradation and flexural strength deterioration, which can be further used for time-dependant reliability analysis and maintenance strategy optimisation for corrosion damaged RC beams.

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TABLES AND FIGURES

List of Captions for Tables and Figures

Table 1. Concrete material properties

Parameter	Symbol	Evaluation	Value	Reference
Compressive strength of cube	f_{cu}		40 MPa (5.80ksi)	
Compressive strength of cylinder	f_{ck}		37.5MPa (5.43ksi)	Eurocode2 [35]
Tensile strength	f_t	$0.39(f_{ck})^{2/3}$	4.4MPa (0.64ksi)	Eurocode2 [35]
Modulus of elasticity	E_c	$11.57(f_{ck}+8)^{0.3}$	36.4GPa (5275ksi)	Eurocode2 [35]
Ultimate crack width	w_u	$\alpha_f \frac{G_F}{0.3 f_{ck}{}^{2/3}}$	0.37mm (0.0146in.)	CEB-FIP [33]
Critical crack width	w_{cr}	$2\frac{G_F}{0.3 f_{ck}{}^{2/3}} - 0.15 w_u$	0.04mm (0.0016in.)	CEB-FIP [33]

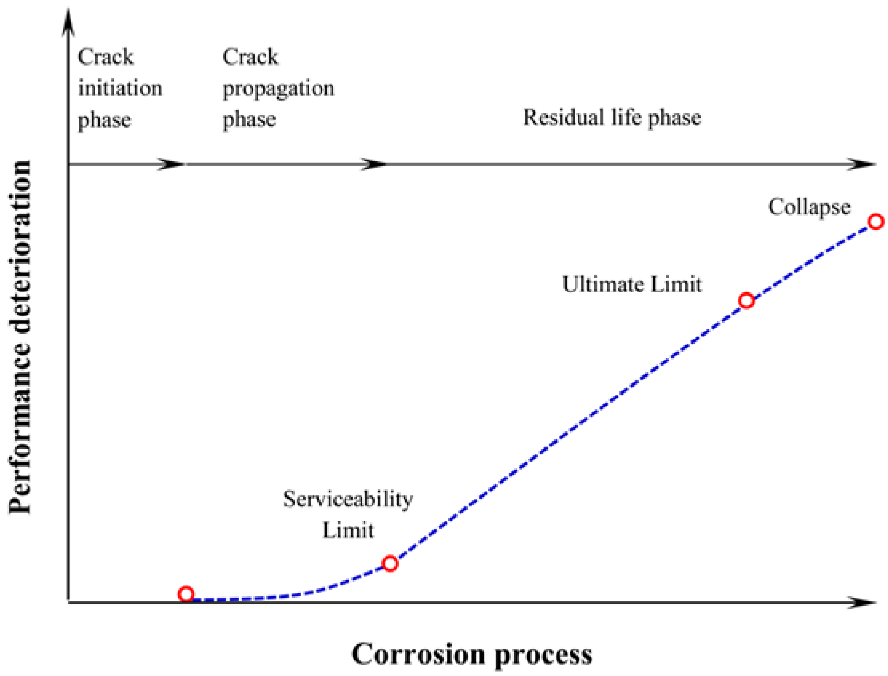

Figure 1. Schematic representation of performance deterioration of RC beams affected by reinforcement corrosion.

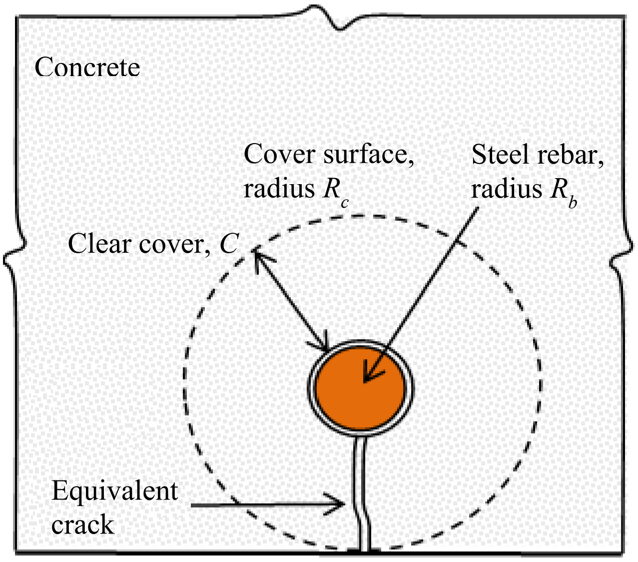

Figure 2. Idealization of cover concrete as thick-walled cylinder model for predicting concrete cracking and residual strength evolution.

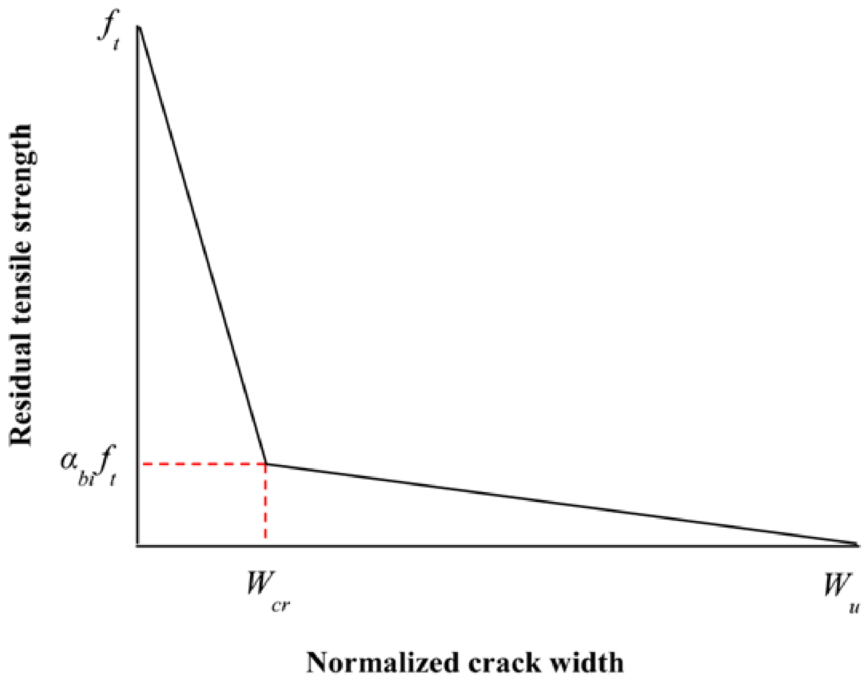

Figure 3. Bilinear tension softening curve for cohesive cracking in the concrete around steel rebar.

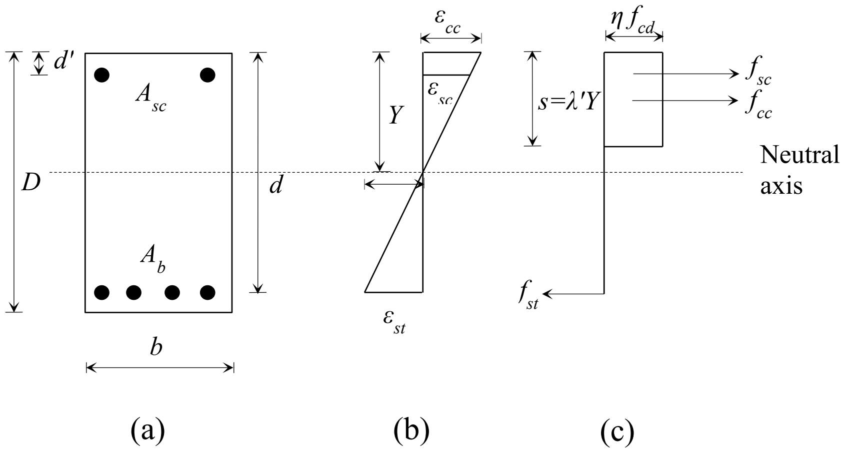

Figure 4. Flexural analysis of a RC beam section: (a) typical cross section of RC beam; (b) strain distribution; (c) equivalent stress distribution.

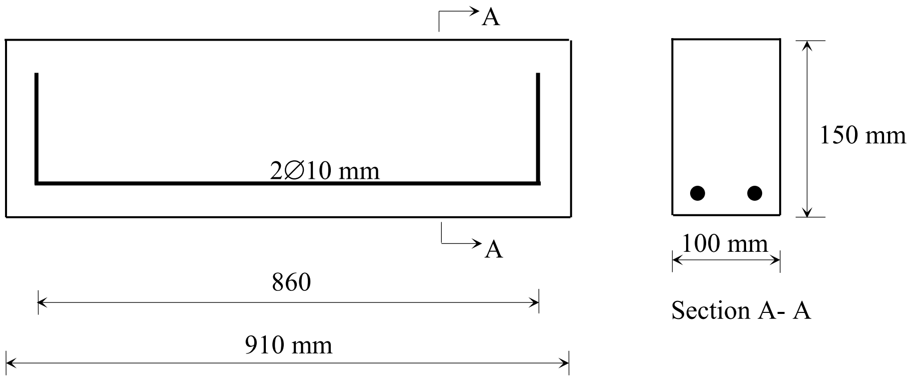

Figure 5. A RC beam specimen and its cross section used in the experimental studies by Mangat and Elgarf [22]. (Note: 1mm = 0.0394in.)

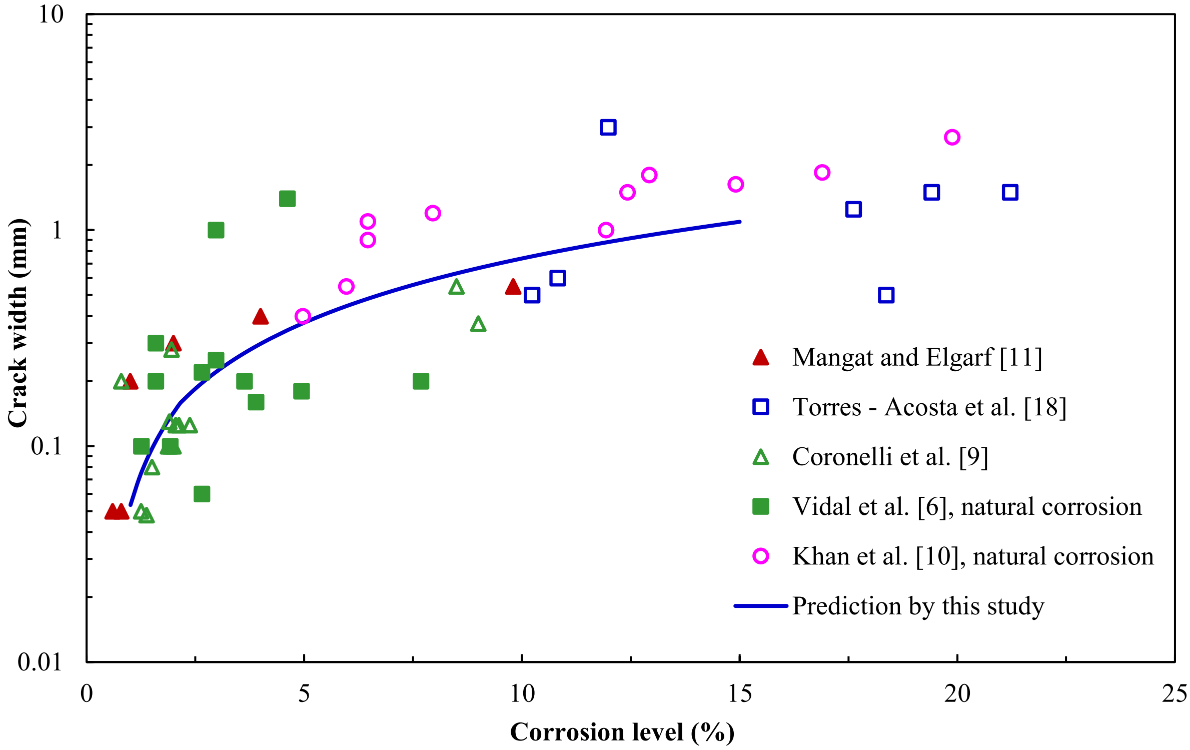

Figure 6. Prediction of equivalent cover surface crack width versus corrosion level, compared with experimental test results available from various sources. (Note: 1mm = 0.0394in.)

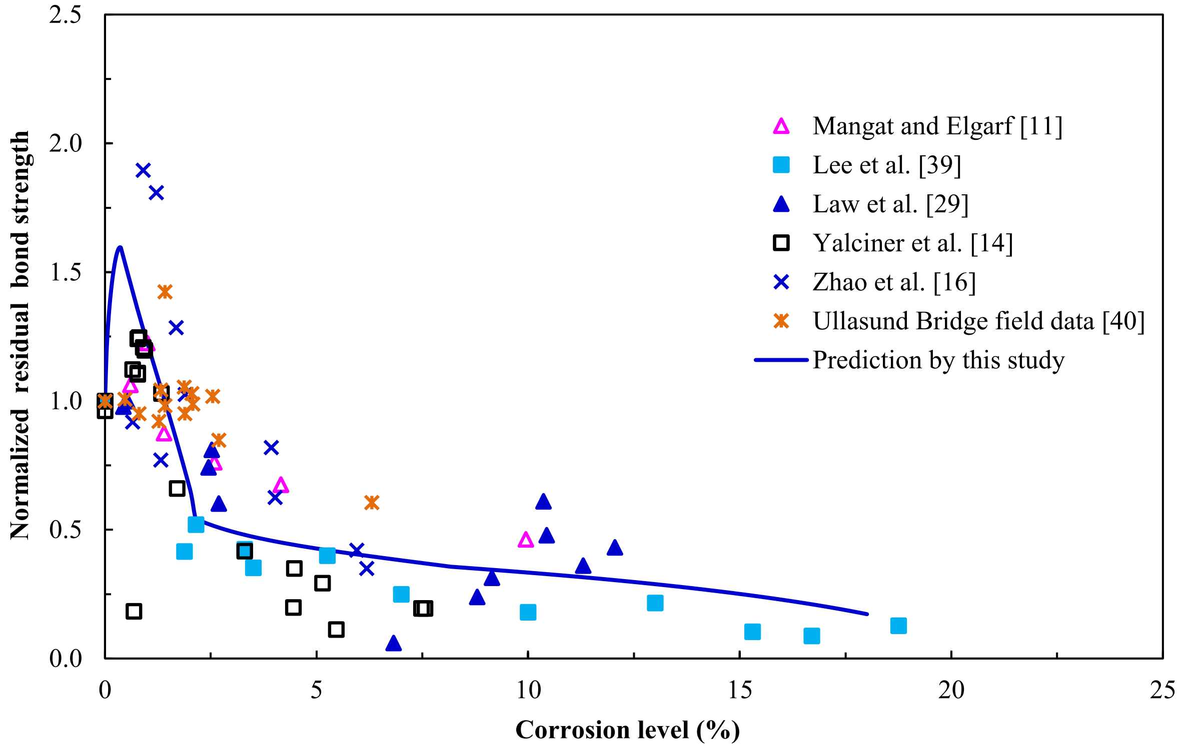

Figure 7. Prediction of normalized residual bond strength versus corrosion level, compared with experimental and field test results available from various sources.

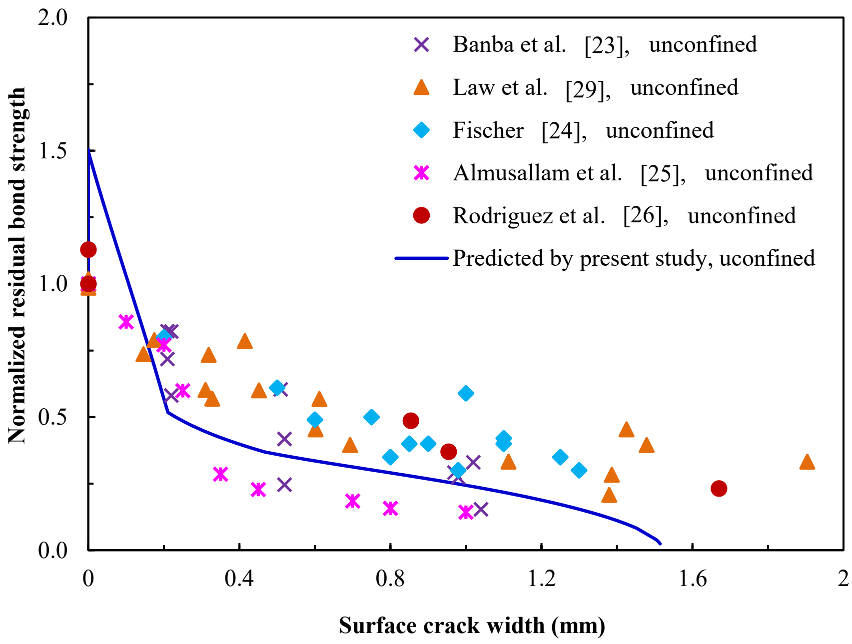

Figure 8. Prediction of normalized residual bond strength versus surface crack width for unconfined specimen, compared with experimental test results available from various sources. (Note: 1mm = 0.0394in.)

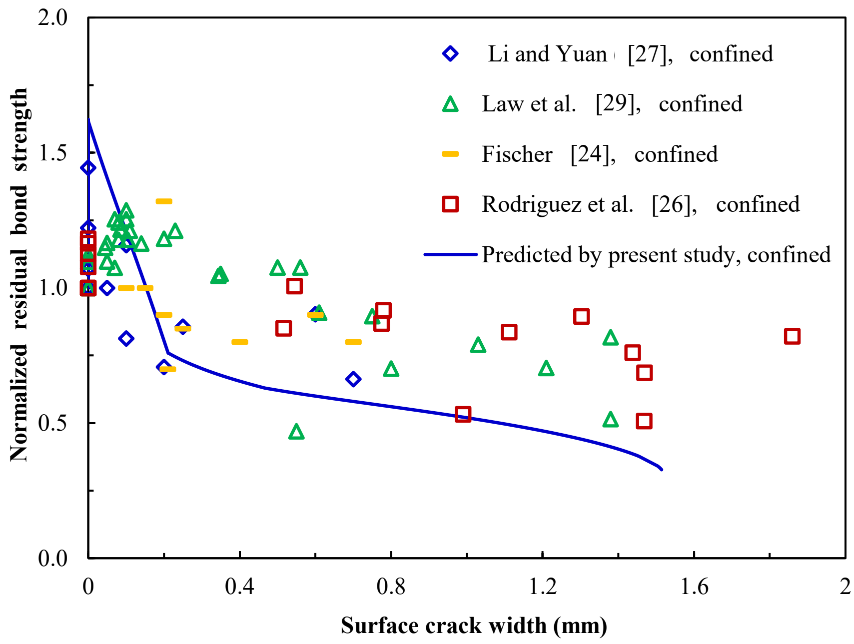

Figure 9. Prediction of normalized residual bond strength versus surface crack width for confined specimen, compared with experimental test results available from various sources. (Note: 1mm = 0.0394in.)

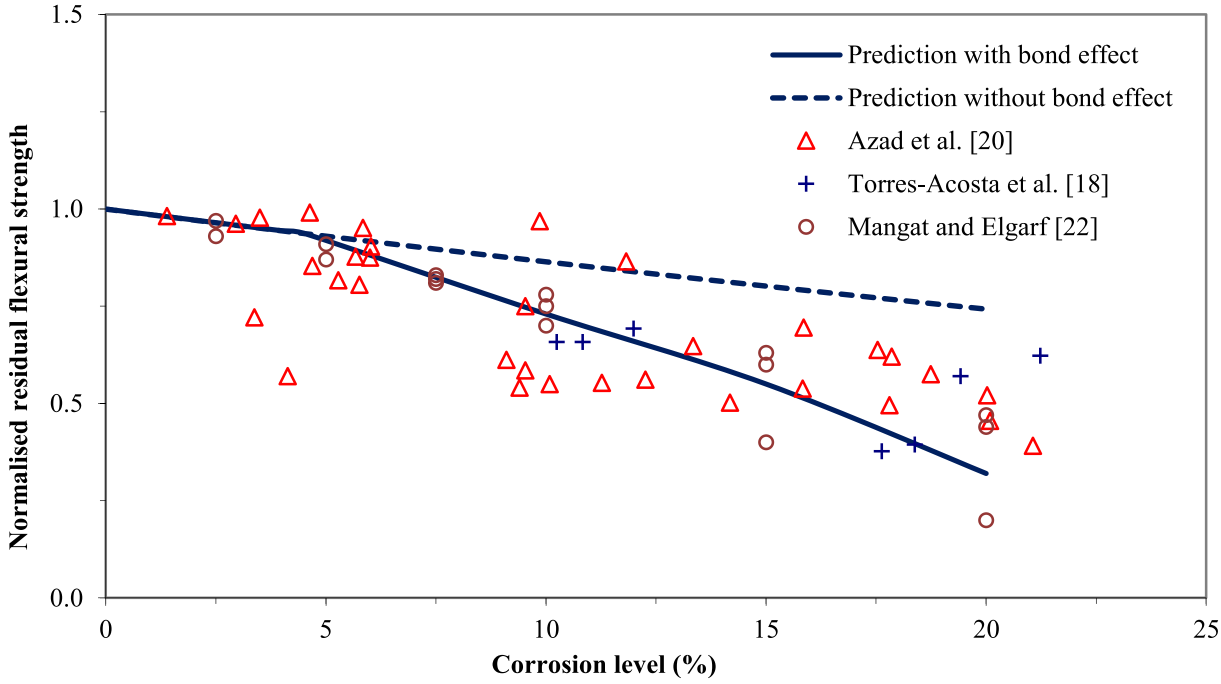

Figure 10. Prediction of normalized residual flexural strength (with and without influence of bond strength loss) versus corrosion level, compared with experimental test results available from various sources.

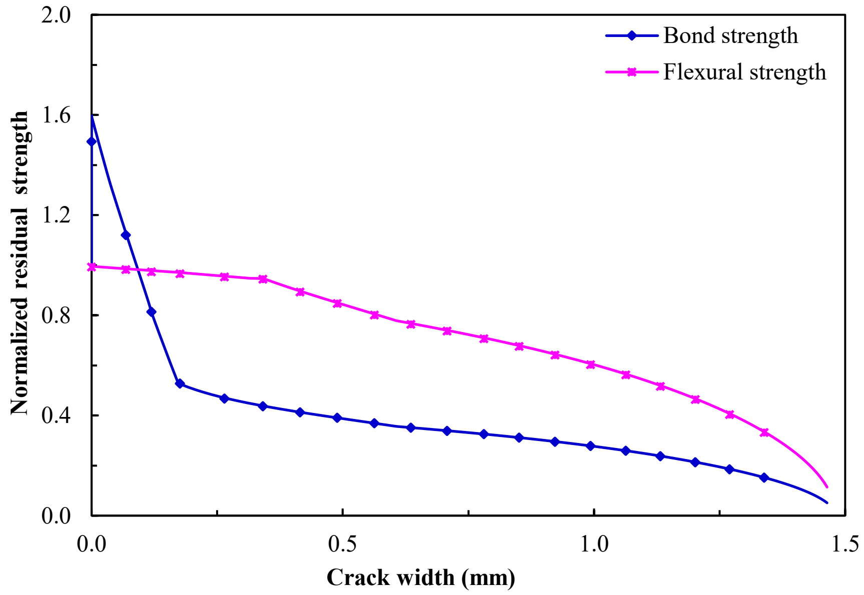

Figure 11. Prediction of normalized bond strength and residual flexural strength versus surface crack width. (Note: 1mm = 0.0394in.)