NUMERICAL SIMULATION OF BEHAVIOUR OF REINFORCED CONCRETE STRUCTURES CONSIDERING CORROSION EFFECTS ON BONDING

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ABSTRACT

Corrosion of reinforcing steel in concrete can alter the interface between the steel and concrete and thus affects the bond mechanism. This subsequently influences the behaviour of reinforced concrete structures in terms of their safety and serviceability. The present paper attempts to develop a numerical method that can simulate the behaviour of reinforced concrete walls subjected to steel corrosion in concrete as measured by their load-deflection relationship. The method accounts for the effects of corrosion on the stiffness, maximum strength, residual strength and failure mode of the bond between the steel and concrete. In the numerical method, the corrosion-affected stiffness and maximum strength of bond are explicitly expressed as a function of corrosion rate. It is found in this paper that the increase in the bond strength due to minor corrosion can increase the load-bearing capacity of the wall and the corrosion-affected reinforced concrete walls exhibit less ductile behaviour compared with the uncorroded ones. The paper concludes that the developed numerical method can predict the behaviour of corrosion-affected reinforced concrete sea walls with reasonable accuracy.

KEYWORDS

Corrosion; Bond-slip; Reinforced concrete walls; Cracking; Finite element modelling.

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INTRODUCTION

Coastal structures, such as sea walls and breakwaters, are essential structural components of sea defences against flooding and erosion. These structures are subjected to increased magnitude and frequency of hydrodynamic actions, larger overtopping flows and increased stresses within the structures. The situation is exacerbated when reinforced concrete (RC) walls deteriorate due to steel corrosion of concrete, which significantly contributes to the premature failure of the walls. One of the significant examples of sea wall failures could be the catostrophi failure of levee system in New Orleans in 2005 which caused 80% of New Orleans flooded. The consequence of failures of sea walls is catastrophic and the impact on the economy is substantial. For example, in the UK at least 3 million people live below 5 meter contour whose life will be at risk if there is a coastal flooding (Purnell 1996).. It is estimated that the total loss in an event of tidal flood at the east coast of the UK could be 6-10 billion US dollars (Purnell 1996).

Corrosion of reinforcing steel in concrete is one of the main causes of premature deterioration of RC structures (Chaker 1992), causing concrete cracking, delaminating and de-bonding. Extensive research has been carried out on the mechanism of steel corrosion and its effect on concrete cracking (Li et al. 2006; Liu and Weyers 1998; Pantazopoulou and Papoulia 2001). However, not many studies have been conducted to evaluate the effect of corrosion on bond mechanism and the consequent effect of deteriorated bond on the structural performance (Amleh and Mirza 1999; Berto et al. 2008; Val and Chernin 2009). When steel bars are corroded in concrete, the chemical adhesion between the bar and concrete basically disappears and the bond is maintained only by mechanical interlock which also degrades. The effect of corrosion on bond strength has been reported to be very significant. For example, 2% loss in diameter of the steel bar can cause 80% reduction in bond strength (Auyeung et al 2000). A literature review (Amleh and Mirza 1999; Auyeung et al 2000; Maaddawy et al. 2003) suggests that most current studies on the effect of corrosion on bond strength focus on experimental research and employ accelerated corrosion tests in an electrical field rather than a more realistic corrosion test produced in hazardous environment. Furthermore, in most research, the steel bar was corroded while the specimens were not loaded (Almusallam et al. 1996; Huang and Yang 1997). This does not represent the real situation of practical structures where both corrosion and applied loads are simultaneous.

To accurately predict the behaviour of RC sea walls, modelling of the bond mechanism, i.e., bond stress-slip relation, between the reinforcing steel and concrete is essential. The influence of bond on the performance of RC structural members has been studied by a number of researchers analytically (Maaddawy et al. 2005), numerically (Kwak and Hwang 2010; Lundgren and Gylltoft 2000) and experimentally (Jung et al. 2002; Kankam 1997; Maaddawy and Soudki 2003). The bond behaviour is commonly modelled by inserting interface elements between the reinforcing steel bar and concrete. It is found (Berto et al. 2008; Coronelli and Gambarova 2004) that almost all the numerical research focuses on macro-scale modelling of the bond. This is because in most experiments the average stress and average slip were measured, and in numerical analysis, it requires substantial computational effort for micro-scale modelling. The simplest and most effective approach to define the interface is to use zero thickness spring-like elements which characterize one-dimensional stress-strain response (Filippou 1986). This means that the bond stress-slip response is considered only in the direction parallel to the reinforcing steel bar.

Consideration of bond deterioration and the degree of corrosion has been widely acknowledged as the most accurate approach to model the behaviour of corrosion-affected RC structures. Coronelli and Gambarova (2004) proposed a modelling procedure in predicting the response of simply supported RC beams subjected to steel corrosion. This procedure accounts for a variety of effects that corrosion may cause, e.g., material property change, bar section reduction, etc. However, it does not consider the change in stiffness of bond due to corrosion and the residual bond strength.

Val and Chernin (2009) numerically modelled the decrease of stiffness of RC beams due to bond loss but only considered the change of maximum bond strength as a result of corrosion. Berto et al. (2008) developed a frictional model and a damage model for bond and applied these two models in simulating a pull-out test of RC specimens. They introduced a scalar damage parameter to account for the loss of bond strength in the full development of slip; such an assumption neglects the fact that minor corrosion can enhance the mechanical performance of the bond and that at different stages of corrosion, the stiffness reduction of bond may be different. Although various models have been developed for different types of RC structures, few models have been proposed on either numerical or experimental simulation of mechanical behaviour of RC sea walls explicitly considering corrosion-induced bond deterioration. Even for simple RC beams, very few studies have been found in literature that contain a comprehensive and accurate consideration of the degree of corrosion on bond behaviour with verification from data produced from realistic corrosion experiment, e.g., natural wetting and drying with salt water.

This study attempts to develop a numerical method to simulate the mechanical behaviour of RC sea walls subjected to steel corrosion. Various effects of corrosion on the RC walls are considered, including reduction of cross-sectional area of the reinforcing steel bar and change in bond mechanism between the steel and concrete. The corrosion-induced change in bond mechanism is modelled in terms of corrosion rate of the steel. A two-node "spring"-like interface element is used to model the corrosion-affected bond mechanism which can account for the nonlinear bond stress softening. In the finite element analysis (FEA), the concrete is assumed as a quasi-brittle material and the cracks in concrete are assumed to be smeared. Whilst the developed method can simulate cracking of the concrete, the effect of cracking on the bond mechanism is not considered. An example is provided for illustration of the developed method and the results are compared favourably with those from the experiments. The derived numerical method can be used as a useful tool in predicting the mechanical behaviour of corrosion-affected RC walls and also other types of

RC structures. Accurate prediction of the performance of corrosion-affected RC structures can help engineers and asset managers ensure their safety and serviceability.

MATERIAL MODEL

The reinforcing steel bar in RC is normally modelled implicitly by adding its "tension stiffness" to the material property of concrete. However, when the bond mechanism between the steel and concrete needs to be modelled, it has to be represented explicitly since the nodes of the elements of bond must be physically attached to the nodes of the steel bar. In this study, RC is modelled as a three-phase material, i.e., concrete, reinforcing steel bar and the bond interface between the bar and concrete.

Since the mechanical behaviour of RC sea walls is predominantly controlled by tensile failure, the compressive property of the concrete is assumed to be elastic, i.e., without the descending part of the typical compressive stress-strain curve of concrete. The damage/cracking of concrete is modelled via a post-peak stress-displacement relation ($\sigma - \delta$) as is shown in Figure 1. Due to the severe mesh sensitivity induced by strain-softening behaviour of concrete, its brittle behaviour is characterized by a stress-displacement relation rather than a stress-strain relation based on the concept of brittle fracture (Hillerborg et al. 1976). The crack is assumed to be smeared in concrete and to occur when the stress reaches the tensile strength $f_t^{'}$ as shown in Figure 1. Therefore, for every material/integration point, a crack/damage is initiated when the tensile strength is reached. However, the material point is not completely separated or damaged until the stress decreases to zero as illustrated in Figure 1.

There are two approaches to model the reinforcing steel in concrete in FEA, i.e., smeared approach and discrete approach. In order to capture the bond-slip behaviour of the interface between the steel and concrete as well as the corrosion effect on it, the steel is explicitly modelled in this paper. For this purpose, a bilinear constitutive relationship is adopted for the steel bar as shown in Figure 2.

The effect of corrosion on reinforcing steel bar is considered by reducing its cross-sectional area. The effect of corrosion on the strength of steel is negligibly small and hence not considered in this paper. The reduction of the cross-sectional area of the steel bar can be calculated as follows (Mangat and Molloy 1992)

$$\mathrm{x} = 11.6 i_{corr} * t \tag{1}$$

where x is the metal loss in radial direction in μm, i_{corr} is the corrosion current density in $\mu A \cdot cm^{-2}$ and t is time in year.

To determine the constitutive relation of the bond, that is, the bond stress-slip curve, FIB (2010) developed a nonlinear model as shown in Figure 3. The curve has a nonlinear increase of bond stress until the maximum bond stress $\tau_{\max}$. This stage refers to the mechanism of local crushing and microcracking due to the penetration of the ribs of the steel bar. The nonlinearity with decreased slope, i.e., stiffness, particularly implies the damage process. The bond stress then keeps constant for certain range of slip normally under confined condition. This stage corresponds to the advanced and continued crushing and tearing off of the concrete. If there is a lack of confinement or subjected to deterioration, this horizontal line will become inclined indicating a splitting failure rather than pull-out failure. After that, the bond stress decreases until a much lower constant level τ_f. This is a typical mechanism for pull-out failure, i.e., the steel bar in concrete is well confined. The splitting failure normally has lower bond strength. For many concrete structures, the load-bearing reinforcing steel is well confined with stirrups and/or sufficient concrete cover. Therefore, pull-out failure can be normally expected without consideration of material deterioration, e.g., steel corrosion. The parameters in Figure 3 are defined in the Model Code of FIB (2010) and summarized in Table 1 where f_{ck} is the characteristic cylinder compressive strength of concrete and c_{clear} is the clear distance between ribs of the steel bar.

In Figure 3, the initial nonlinear increase in the bond stress-slip relationship can be expressed as follows (FIB 2010)

$$\tau = \tau_{\max}\left(s / s_1\right)^{\alpha} \tag{2}$$

where s is the slip, and s_1 are α constants defined in Table 1.

EFFECT OF CORROSION ON BOND

The bond between the reinforcing steel and concrete plays a vital role in the mechanical behaviour of RC structures (Sanchez et al. 2010). In most of the structural design codes, the bond is assumed to be intact without consideration of any bond loss induced by corrosion and/or cracking. In practice, when a RC structure is subjected to steel corrosion, the corrosion products will normally alter the condition of the interface between the reinforcing steel and concrete and hence the frictional components of the bond. This can significantly reduce the bond strength at a later stage of corrosion even though some experimental results (FIB 2000) suggest that the bond strength can be enhanced slightly under minor corrosion – up to 4% degree of corrosion by mass loss (Almusallam et al. 1996; Rodriguez et al. 1994) – due to the increase of the friction between the steel bar and concrete. In addition to the bond strength, corrosion can affect all the other parameters that define the constitutive relationship of the bond, i.e., Figure 3, as discussed below.

Maximum bond strength. The bond stress reaches its maximum value $\tau_{\max}$ usually at the point of time that concrete cover is cracked (Almusallam et al. 1996; Val and Chernin 2009). In addition to the subsequent "softening" behaviour, the maximum bond strength itself is subjected to change due to steel corrosion. Based on the experimental results, Almusallam et al. (1996) proposed an exponential deterioration of the maximum bond strength as shown in Figure 4. The normalized bond strength refers to the ratio of the corrosion-affected bond strength $\tau_{\max}^{c}$ to the bond strength without corrosion, i.e., $\tau_{\max}^{c} / \tau_{\max}$.

In Figure 4, the maximum bond strength is proposed to increase exponentially up to a degree of corrosion of 4% (by mass loss) and decrease exponentially to a relatively constant value (about 10% of its original strength) after about 20% of corrosion. Based on the data produced in Almusallam et al. (1996), an analytical form of the normalized bond strength can be expressed as follows.

$$\frac{\tau_{\max}^{c}}{\tau_{\max}} = 0.9959e^{0.0041w} + 0.0069e^{0.7858w} \qquad \text{for } w \le 4(\%) \tag{3a}$$

$$\frac{\tau_{\max}^{c}}{\tau_{\max}} = 9.662e^{-0.5552w} + 0.1887e^{-0.0069w} \qquad \text{for } 4(\%) < w \le 80(\%) \tag{3b}$$

where w is the degree of corrosion by mass loss of the original steel.

The amount of corrosion products $W_{rust}(t)$ can be related to the corrosion rate of the reinforcing steel bar, i.e., (Liu and Weyers 1998)

$$W_{rust}(t) = \sqrt{2\int_{0}^{t} 0.105(1/\alpha_{rust})\pi D i_{corr}(t)dt} \tag{4}$$

where i_{corr} is the corrosion current density in $\mu A/cm^2$, which is widely used as a measure of corrosion rate. α_{rust} is the molecular weight of steel divided by the molecular weight of corrosion products. It varies from 0.523 to 0.622 according to different types of corrosion products (Liu and Weyers 1998). In Equation (4), D is the diameter of the steel bar in *mm* and t is time in *year*.

The corrosion degree w can be related to the amount of corrosion products $W_{rust}(t)$ (in mg/mm). A simplified form of this relationship can be expressed as follows,

$$w = \frac{4\times10^5 \alpha_{rust} W_{rust}}{\pi\rho_{st} D^2} \tag{5}$$

where ρ_{st} is the density of steel.

Substituting w in Equation (3) and $W_{rust}(t)$ in Equation (5), the corrosion-affected bond strength becomes a function of the corrosion rate, as follows:

$$\frac{\tau^c_{\max}}{\tau_{\max}} = 0.9959e^{\frac{0.75\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t)dt}}{\sqrt{\pi}\rho_{st}D^2}} + 0.0069e^{\frac{144\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t)dt}}{\sqrt{\pi}\rho_{st}D^2}} \quad \text{for } w \le 4(\%) \tag{6a}$$

$$\frac{\tau^c_{\max}}{\tau_{\max}} = 9.662e^{\frac{-0.25\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t)dt}}{\sqrt{\pi}\rho_{st}D^2}} + 0.1887e^{\frac{-0.0032\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t)dt}}{\sqrt{\pi}\rho_{st}D^2}} \quad \text{for } 4(\%) < w \le 80(\%) \tag{6b}$$

Stiffness of bond. The stiffness of bond between corroding steel and concrete varies depending on the degree of corrosion. Unfortunately almost all the current research focuses on the corrosion effect on the bond strength (Auyeung et al 2000; ; Cabrera and Ghodussi 1992). A literature review (Almusallam et al 1996; Auyeung et al 2000) suggests that there is no analytical model existing for corrosion effect on the stiffness of bond. Some researchers believe that the expected possible reduction of the bond stiffness due to corrosion leads to a small decrease of stiffness of the global structure (Val and Chernin 2009). However, based on the test results (Almusallam et al. 1996), the variation of stiffness for different degree of corrosion can be very large. For the sake of accurate prediction of the bond behaviour, the corrosion-induced change in bond stiffness needs to be taken into account. Based on Equation (2) for the case of no corrosion, the corrosion-affected bond stress τ^c prior to its maximum value can be proposed as follows,

$$\tau^c = \tau_{\max}^c s^{0.4} \tag{7}$$

The stiffness of the corrosion-affected bond is dependent on its maximum strength. By substituting $\tau_{\max}^c$ of Equation (6a), Equation (7) can be rewritten as follows,

$$\tau^c = \tau_{\max} s^{0.4} \left(0.9959 e^{\frac{0.75\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t) dt}}{\sqrt{\pi}\rho_{st} D^2}} + 0.0069 e^{\frac{144\sqrt{\int_0^t \alpha_{rust} D i_{corr}(t) dt}}{\sqrt{\pi}\rho_{st} D^2}} \right) \quad \text{for } w \le 4(\%) \tag{8}$$

τ^c for $4(\%) < w \le 80(\%)$ is similar. It can be seen that τ^c has considered a number of factors, including slip s, corrosion rate $i_{corr}(t)$, concrete compressive strength (in terms of $\tau_{\max}^c$), diameter of the bar, etc.

Failure mode of bond. Without steel corrosion, the bond can fail in a manner characterised by splitting of cover concrete or pull-out, depending on the extent of confinement. Once the steel is corroded, due to the fact that corrosion will mainly reduce the confinement to the steel, most of the relevant research assumes that the corrosion-affected bond failure is caused by splitting mechanism

(Almusallam et al. 1996; Coronelli and Gambarova 2004). There might be an argument that minor corrosion may increase the confinement and the pull-out failure may still be likely. However, no experimental evidence has been found in the literature to support this argument. It is therefore reasonably accepted in this study that splitting failure is considered as the dominant mode for corrosion-affected bond failure.

Residual bond strength. With the existence of stirrups, bond strength does not normally reduce to zero but to certain constant level τ_f. The corrosion can reduce its magnitude, e.g., τ_f^c. Based on very few experimental sources (Almusallam et al. 1996; Auyeung et al. 2000), it is assumed that τ_f will be degraded to 15% of the maximum bond strength (compared to 40% for uncorroded case (FIB 2010)) for all corrosion conditions.

FORMULATION OF FINITE ELEMENT MODEL

RC sea walls can be modelled as 2-D plane strain problem since the third dimension is sufficiently large compared to the other 2 dimensions. In the plane, two reinforcing steel bars (one in tension and the other in compression) are embedded in the concrete as shown in Figure 5 (a). Compressive reinforcing steel bar is assumed to have perfect bond with the concrete while the bond for the tensile steel bar degrades. Therefore, interface elements, which are able to implement the bond stress-slip behaviour, are adopted between the tensile steel bar and concrete only. As such, the geometry can be setup as shown in Figure 5 where (a) illustrates the sea wall in FE mesh and (b) shows the interface elements between the tensile steel bar and concrete. The load is applied to the top from left to right.

Finite element package ABAQUS is employed to carry out the stress analysis. The interface element used in FEA is a "line" element named "translator" in ABAQUS, as illustrated in Figure 6. The element has 2 nodes **a** and **b**, and each of them has the normal degrees of freedom (DOF), i.e., 3 DOF for 2D problem for each node. The local coordinates of nodes **a** and **b** are also shown in Figure 6. These 2 nodes are connected to concrete and steel bar respectively and physically represent the bond. The configuration of the interface element is set such that only the displacement in x-axis is enabled while all the other degrees of freedom for these two nodes are relatively restrained to each other. This means that only relative movement (slip) between nodes a and b is allowed while all the other degrees of freedom of node **a** are equal to those of node **b**. The x-axis of local coordinate system can be transformed to accommodate various needs for slip directions, e.g., slip expected in the vertical direction.

The advantage of the chosen element against other elements available in ABAQUS is its capability of modelling the deterioration of bond. The damage of bond is modelled in the interface elements in terms of the full corrosion-affected bond stress-slip relationship. Damage initiation marks the beginning of degradation or the damage of bond at a point. The damage is assumed to initiate when the bond stress reaches the maximum bond strength $\tau^c_{\max}$. After the damage is initiated, the bond stress of the interface element immediately softens as for the splitting failure mode. The softening curve follows a straight line to about 15% of its peak stress. To account for the residual bond stress τ^c after its peak value, a damage parameter β (between 0 and 1) is introduced and defined as follows (also shown in Figure 7):

$$\tau^c = (1-\beta)\tau^c_{\max} s \qquad (0 \le \beta \le 1) \tag{9}$$

As discussed above concrete is considered to be cracked once the tensile stress in concrete (maximum principal stress for current problem) reaches the tensile strength f_t' but does not lose all of its strength since the concrete is assumed to be quasi-brittle. The tensile stress in concrete will gradually decrease to zero. In this study, the cracks of concrete are assumed to be smeared in the concrete, with reducing level of the tensile stress while the strain increases. The crack propagation of concrete is controlled by energy dissipation, as shown in Figure 8. A scalar damage (cracking) parameter γ is introduced as follows (Yang and Li 2011):

$$\gamma = \frac{G_r}{G_f - G_e} = \frac{\int_{\delta_0}^{\delta_r} f(\delta) d\delta}{G_f - \frac{f_t^{'}\delta_0}{2}} \tag{10}$$

where $f(\delta)$ is the softening function, G_r is the energy release rate after peak stress, and G_e is the elastic energy release rate prior to peak stress.

Four-node plane strain quadrilateral elements with four integration points are used for both concrete and steel. Very fine mesh is adopted to capture the cracking/strain softening behaviour of concrete. Some small value of viscous quantity is normally required for Newton-Raphson method in FEA. Its determination is based on trial and error but normally should be between 1e-3 and 1e-5. Values beyond this may incur risk of comprising results.

WORKED EXAMPLE AND VERIFICATION

To demonstrate the application of the derived FE model to simulating mechanical performance of RC sea walls subjected to steel corrosion, an example is undertaken. The dimension of the example sea wall is taken as 1000×2000×150 mm. The values of basic variables used in the example are presented in Table 2. Load is applied at the top of the wall. With these values the load-deflection relation of the walls without and with steel corrosion can be obtained from the FEA and shown in Figure 9 with three degrees of corrosion, i.e., 0%, 4% and 8%. From Figure 9, it can be seen that at about 50 mm deflection, the load for the walls with both degrees of corrosion (4% and 8%) starts to decrease whilst the load for the wall without corrosion continues to increase, suggesting that corrosion has caused bond deterioration. It can also be observed that after about 100 mm deflection, the load-bearing capacity of the wall with corrosion is smaller than that without corrosion, with that of 8% corrosion the lowest, suggesting the higher degree of corrosion, the greater the bond loss. It is clear that the corrosion with the degree greater than 4% can reduce the load-bearing capacity of RC walls. With minor corrosion (<4%), the load carrying capacity of the wall is more or less the same, regardless of the corrosion. However the load-bearing capacity starts to "degrade" at much smaller value of deflection compared to that of uncorroded sea wall. It is also clear that the behaviour of corroded walls (both 4% and 8% corrosion) is slightly less ductile than that of the uncorroded ones, with regards to the ability to soften. This can be attributed to the default failure modes in FEA, i.e., splitting failure mode is assumed for the corrosion-affected bond deterioration while pull-out failure mode is adopted for uncorroded bond.

Based on the model of corrosion rate i_{corr} proposed in Li (2000) and shown in Table 2, the bond strength (normalised by the maximum value of bond strength) can be directly expressed as a function of time from Equation (6) which is shown in Figure 10. It can be seen that the corrosion-affected bond strength slightly increases to nearly 1.2 of its original bond strength, followed by a nonlinear decrease at about 40 years. This suggests that corrosion initially increases the bond strength but later reduces it, the extent of which largely depends on the corrosion rate.

To verify the above numerical results, data are obtained from experiments which were conducted on specimens with structurally significant size (i.e., a representative of real structures) to investigate steel corrosion in concrete and its effects on structural performance. The dimension of specimens was 1000×2000×150 mm. There were a total of 22 specimens loaded to failure at different degrees of corrosion. The typical cross-section of the specimen is shown in Figure 11.

Three concrete covers were used, i.e., 30 mm, 40 mm and 50 mm with the ratio of concrete cover to steel bar diameter being 3, 4 and 5 respectively. The material properties of concrete and reinforcing steel are listed in Table 3. To simulate the working conditions of sea walls, a large environmental chamber was constructed to house the specimens. As schematically shown in Figure 12, the specimens were wet by salt water spray in the chamber and dried alternatively. A pair of test specimens was loaded through two rods at top of the wall so that the specimen walls were subjected to simultaneous corrosion and service loading. Service load applied to the wall is the design load calculated according to UK Code 8110. A detailed description of the test program can be found in Li et al (2005).

All specimens were under the accelerated conditions for corrosion process which was achieved by intensifying (and hence shortening) the drying period of wetting and drying cycles, and by spray of salt water directly to the walls, especially to the cracked surface of the walls. An intense drying period was imposed to let saturated concrete dry in time to take in oxygen more effectively. Otherwise the concrete would remain saturated before wetting again. A typical day of test conditions is shown in Table 4 (Li 2000). The specimens were placed in the environmental chamber for corrosion for 8, 16 and 24 months respectively at end of which, they were taken out of the chamber for ultimate load testing in order to obtain measurements over time.

The key measurement is corrosion rate which was measured by linear polarization resistance throughout the testing period. The results of i_{corr} for specimens of 30 mm, 40 mm and 50 mm concrete cover are shown in Figure 13. It should be noted that, in Figure 13, the testing time has been translated into the real natural time t by multiplying an acceleration factor proposed in Li (2000). The value of i_{corr} is the averaged results of about 10 measurements taken at the point of time. Based on the data obtained, an expression of the corrosion rate (i_{corr}) as a function of time can be determined as follows:

$$i_{corr} = 0.0652 \times t + 1.0105 \quad \text{for cover of 30 mm} \tag{11a}$$

$$i_{corr} = 0.0369 \times t + 0.9472 \quad \text{for cover of 40 mm} \tag{11b}$$

$$i_{corr} = 0.0458 \times t + 1.8498 \quad \text{for cover of 50 mm} \tag{11c}$$

where t is (calibrated) time in year.

Equations (11) can be used in the numerical model to calculate the reduction of the cross sectional area and the change of constitutive relationship of the bond, from which the load-deflection relation for the sea wall can be obtained from the FEA for different concrete covers. The results are then compared with those from the experiments as shown in Figure 14. It can be seen that the numerical results of load-deflection relation of the specimen wall agree well with the experimental results, especially in the initial stage. It has been observed that the specimen of 50 mm cover can sustain highest load compared to those of 40 mm and 30 mm covers. This is probably because the extra thickness in covering the reinforcing steel provides additional confining pressure to the steel bar which increases the bond strength. The enhanced bond strength subsequently increases the global mechanical performance of the specimen wall.

The effects of the degree of corrosion on the load-deflection relation for test specimens are also examined. Based on the calibration method proposed in Li (2000), the accelerated time for corrosion can be translated to natural time for corrosion, using an accelerated factor of 50, i.e., 1 cycle = 50 days. For example, 8 months accelerated corrosion (2 cycles per day) is equivalent to 34 years in natural time, from which and according to Equation (11), the averaged corrosion rates at year 34 are 2.12 $\mu A \cdot cm^{-2}$ for 30 mm cover, 1.57 $\mu A \cdot cm^{-2}$ for 40 mm cover and 2.63 $\mu A \cdot cm^{-2}$ for 50 mm cover. Substituting these corrosion rates into Equation (1), the diameter losses of steel bar are calculated to be 0.84 mm for 30 mm cover, 0.62 mm for 40 mm cover and 1 mm for 50 mm cover respectively. These reduced diameters of the steel bars are used in the FEA. Four points in time, i.e., 20 years, 34 years, 50 years and 70 years are chosen to evaluate the corrosion effect. The results are presented in Table 5.

With the modified bond mechanism and the reduction of the cross sectional area of steel bars, the load-deflection relationship of the specimens with corrosion at 8 months (testing time) is re-determined numerically and shown in Figure 15. Also shown is the comparison with the experimental results. Again very reasonably good agreement has been achieved. The difference in load-deflection curves might be related to the slightly different boundary conditions for the numerical model and the experiment. For example, the numerical model assumes that the sea wall is fixed at the bottom, while in the experiment the sea wall can slightly rotate which caused an additional very small deflection under given load. Due to the numerical difficulties in obtaining converged results of FEA that involves excessive materials damage, the numerical simulation does not normally go as far as the experiments, i.e., the wall fails at smaller deflection for the numerical modelling compared to that for the experiments. Also from Table 5, the degree of corrosion for specimens 30 mm, 40 mm and 50 mm concrete cover is 2.67%, 2.30% and 2.97% respectively (all < 4%). The nominalized bond strength therefore has increased 1.06, 1.05 and 1.08 respectively. This partially explains why the specimens with 50 mm concrete cover have the highest load-bearing capacity, followed by specimens with concrete covers of 30 mm and 40 mm. It is of interest to note that the load-bearing capacity depends more on bond strength than on concrete covers which makes sense according to RC theory.

CONCLUSIONS

In this paper a numerical method to simulate the mechanical behaviour of corrosion-affected reinforced concrete sea walls has been proposed. Both slips between the reinforcing steel and concrete for cracked concrete and the effects of steel corrosion on the bond-slip relation have been considered explicitly. Models for corrosion-affected maximum bond strength and bond stiffness have been developed as a function of corrosion rate. In the numerical method, the bond failure strength and failure mode can be modified based on the degree of steel corrosion. It has been found that the load-deflection relationship predicted by the proposed numerical method is in good agreement with that from the experiments. It has been also found that the increase in the bond strength due to minor corrosion can increase the load-bearing capacity of the wall, and that the steel corrosion can alter the flexural stiffness of sea walls depending on the degree of corrosion. Moreover, the corrosion-affected concrete walls have been found to exhibit less ductile behaviour compared with uncorroded concrete walls. It can be concluded that the proposed numerical method can predict the mechanical behaviour of corrosion-affected reinforced concrete sea walls with reasonable accuracy. Accurate prediction of the performance of corrosion-affected reinforced concrete structures can help engineers and asset managers ensure their safety and serviceability.

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