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Structural capacity assessment of corroded RC bridge piers

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A new numerical model is developed that enables simulation of the non-linear flexural response of reinforced concrete (RC) components and sections with corroded reinforcement. The numerical model employs a displacement-based beam-column element using the classical Hermitian shape function. Material non-linearity is accounted for by updating element stiffness matrices using the moment-curvature response of the element section considering uniform stiffness over the element. The cover concrete strength is adjusted to account for corrosion-induced cover cracking and the core confined concrete strength and ductility are adjusted to account for corrosion-induced damage to the transverse reinforcement. The numerical model is validated against a benchmark experiment on a corroded RC column subject to lateral cyclic loading. The verified model is then used to explore the impact of corrosion on the inelastic response and the residual capacity of corroded RC sections. The results show that considering the effect of corrosion damage on RC sections changes the failure mode of RC columns.

Notation

A	total area of steel and concrete
A'	pitted cross-section area of corroded bar
A_{ave}	average reduced cross-section area of corroded bars
A_s	area of reinforcing steel
b	breadth of the column section
D	vertical bar diameter
d	effective depth of referencing steel
E_h	hardening modulus of steel
E_s	elastic modulus of steel
I'_{min}	minimum second moment of area of corroded bars
K	mean value of pitting coefficient of minimum second moment of area
L	tie spacing
M_z	applied moment on a section
N	applied axial force on a section
S_{M_z}	longitudinal stress in section due to bending moment M_z
S_N	longitudinal stress in section due to axial force N
β_c, β_s	pitting coefficients
γ	mean value of area pitting coefficient
δ	shape parameter
ε	strain
ε_0	average strain of the section
ε_{co}	strain at peak compressive stress (σ_c)
ε_p	plastic strain ($=\varepsilon-\varepsilon_y$)
ε_u	ultimate strain of uncorroded referencing steel
ε'_u	ultimate strain of corroded reinforcing steel
ε_{utie}	fracture strain of tie/spiral reinforcement

ε_y	yield strain
ε_1	strain at bottom of section
ε_2	strain at top of section
η	coefficient related to bar roughness and diameter
κ_z	curvature
λ_p	bar buckling parameter
μ	hardening ratio ($=E_h/E_s$)
ρ_{sc}	volumetric ratio of confinement reinforcement
ρ_1	initial tangent of post-buckling response curve
ρ_2	rate of change of tangent
σ_c	stress in concrete
σ_s	stress in steel
σ_y	yield stress of uncorroded bar
σ_{ytie}	yield strength of horizontal tie reinforcement
σ'_y	yield stress of corroded bar in tension
σ'_{yc}	yield stress of corroded bar in compression
σ^*	asymptotic lower stress limit of post-buckling curve
ψ	percentage mass loss due to corrosion

1. Introduction

Among the different deterioration mechanisms, corrosion of reinforcing steel is the most common reason for the premature deterioration of reinforced concrete (RC) structures in a chloride-laden environment. This is an important but untimely threat to the safety of historic engineering structures. This premature loss of structural capacity has serious economic cost implications in developed countries. In the UK, the Department of Transport estimated that salt-induced corrosion damage on motorway and trunk road bridges totalled

£616.5 million in England and Wales alone (Wallbank, 1989) and these bridges represent only about 10% of the total bridge inventory in the country. The American Transportation Research Board also reported that there is US\$ 150 billion worth of corrosion damage on their interstate highway bridges caused by de-icing and sea salt induced corrosion (Hida *et al.*, 2010). In 2013, the American Society of Civil Engineers reported that about 11% of US bridges were classified as structurally deficient and 24.4% defined as functionally obsolete (ASCE, 2013). They estimate the current cost to repair or replace deficient bridges only to be about US\$ 76 billion; this sum is up from 2009 when the total cost was estimated to be US\$ 71 billion (ASCE, 2013).

Corrosion leads to loss of the steel within the cross-section and a weakening of the bond and anchorage between concrete and reinforcement. This directly affects structural serviceability and strength. Many corroded bridges are also located in regions of high seismic activity. These structures experience dynamic/cyclic loading due to earthquakes over their service life. Therefore, corrosion can significantly increase the seismic risk of deteriorating structures. This increased vulnerability may be seen at all performance levels and so can increase the whole lifecycle cost of the structure. Moreover, the current design approach allows RC structures to dissipate energy during large earthquake events by utilising plastic hinges. Among RC components, bridge piers are the most vulnerable components in earthquakes due to the simple structural form of bridges.

Several researchers have investigated the effect of corrosion on the stress-strain behaviour of reinforcing bars in tension (Apostolopoulos, 2007; Apostolopoulos *et al.*, 2006; Cairns *et al.*, 2005; Du *et al.*, 2005a, 2005b). Kashani *et al.* (2013a, 2013b, 2013c, 2014, 2015a, 2015b, 2015c) conducted comprehensive experimental and computational studies on the inelastic behaviour of isolated corroded reinforcing bars. These included the impact of corrosion on inelastic buckling and degradation due to low-cycle fatigue. The results obtained were in good agreement with the results of other researchers studying the cyclic behaviour of RC components.

In recent years, several researchers have studied the seismic vulnerability and fragility analysis of corroded RC bridges (Alipour *et al.*, 2011; Choe *et al.*, 2008; Ghosh and Padgett, 2010). They have investigated the effect of reinforcement corrosion on the non-linear behaviour and response of RC bridges subject to seismic loading. These studies used non-linear fibre-based finite-element analyses (PEER, 2014), but used very simple uniaxial material models to model the impact of corrosion on the stress-strain behaviour of reinforcing steel. In most cases, the corrosion damage was limited only to the reinforcing steel by considering an average reduced area and/or reduced yield strength. Furthermore, these studies ignored the

impact of corrosion on ductility loss, inelastic buckling of vertical reinforcement and corrosion-induced damage to cover and core confined concrete.

In this paper a computational technique is developed that enables simulation of the non-linear flexural response of RC components with corroded reinforcement. The model employs a new uniaxial material model for corroded reinforcing steel. This model simulates the stress-strain behaviour of corroded reinforcing steel with the effect of inelastic buckling (Kashani *et al.*, 2015a). The computational model is validated against a benchmark test on a representative corroded RC column. The results of experimental testing of a corroded column and verification of the computational model are reported in this paper. Finally, the verified model is used to investigate the effect of corrosion on the non-linear response and residual capacity of corrosion-damaged RC sections.

2. Experimental programme

A comprehensive set of experimental tests was proposed to investigate the effect of corrosion on the non-linear behaviour of RC components. Firstly, the influence of corrosion on the non-linear stress-strain behaviour of reinforcing bars under monotonic (tension and compression, including the effect of inelastic buckling) and cyclic loading was explored. Experimental testing was conducted on about 150 corroded reinforcing bar test specimens, followed by numerical modelling of the specimens. The outcomes of these studies are reported elsewhere (Kashani *et al.*, 2013a, 2013b, 2013c, 2014, 2015b, 2015c). Using the experimental results, an advanced uniaxial material model for simulation of the stress-strain behaviour of corroded reinforcing bars was developed (Kashani, 2014; Kashani *et al.*, 2015a). This new material model was used in the development of a computational technique to model the non-linear behaviour of RC sections and components, and this is reported in Section 3 of this paper.

The second part of this research programme was a benchmark reaction wall test on a prototype corroded RC column. The results of this experiment were used for validation of the computational model. Section 2.1 presents the details and results of this experiment.

2.1 Reinforced concrete column specimen

A RC column 250 mm by 250 mm in cross-section and 2500 mm high (height above the foundation) was designed to EC2 (BSI, 2004) criteria. The column section contained eight 12 mm diameter vertical bars with 8 mm diameter horizontal tie reinforcement. The tie reinforcement was spaced at 50 mm up to 800 mm above the foundation and thereafter at 150 mm. The column and foundation were cast separately and, after completion of the corrosion process, the column was cast into the foundation. The cover concrete was 25 mm and

Figure 1. Reinforcement details of RC column test specimen (dimensions in mm)

the maximum aggregate size of the concrete was 10 mm. Figure 1 shows the details of the column test specimen and Tables 1 and 2 summarise the mechanical properties of the steel and concrete used in this test specimen

It should be noted that this column was not intended to represent a full-scale bridge pier, but is representative of typical RC columns designed to EC2 to investigate the impact of corrosion. The dimensions of the column were chosen so that the column was reasonably large but also reasonably easy to break using the available high-performance actuators. A larger size column could have been chosen, but this would have required a larger actuator for structural testing. In terms of slenderness, an attempt was made to make the column flexurally governed

	Reinforcement type	
	8 mm (B8)	12 mm (B12)
Yield strength, f_y : MPa	510	540
Modulus of elasticity, E_s : MPa	194 881	212 099
Yield strain, $\epsilon_y = f_y/E_s$	0.00261	0.00254
Ultimate strength, f_u : MPa	645	616
Ultimate strain at maximum stress, ϵ_u	0.04660	0.06033
Strain ratio, ϵ_u/ϵ_y	17.85	24.42
Strength ratio, f_u/f_y	1.27	1.18
Total elongation at maximum force, λ : %	4.66	6.03
Total elongation at failure, λ_f : %	7.01	16.1
Unit mass, m : kg/m	0.396	0.874

Table 1. Mechanical properties of uncorroded reinforcement

w/c	Maximum aggregate size: mm	Maximum strength: MPa
0.45	10	30

Table 2. Properties of concrete

in order to avoid any significant non-linear shear deformation. Material testing showed that the scale did not affect the stress–strain behaviour of the materials used in the experiment because they were still within the normal range of materials used in typical RC construction. It should be noted that the stress–strain behaviour of reinforcement was not affected by the bar diameter and cube tests showed that the stress–strain behaviour of the concrete was not affected by the reduced aggregate size (see Table 2). Therefore, scaling did not affect the results.

2.2 Accelerated corrosion procedure and cyclic reaction wall test

The RC column was first subjected to an accelerated corrosion process by applying an anodic current of specified intensity and time. This comprised an electrochemical circuit using an external power supply. The reinforcing bars acted as an anode in the cell and the external material (stainless steel) acted as the cathode. Only the part of column that would be immediately above the foundation (800 mm above base level) was immersed in a tank containing 5% sodium chloride solution. A data acquisition system was set up to monitor the current and voltage applied to the column during the test. After completion of the accelerated corrosion the column was cast into the foundation block.

The predicted percentage mass loss using Faraday's law of electrolysis (Kashani *et al.*, 2013a) was 15% after applying an average current of 3 A for 4 months. The power supply was set to 3 A current, but monitoring data showed that an average of 2.15 A was applied over the 4 months. The actual mass loss of corroded reinforcement inside the concrete was calculated by measuring the actual mass of corroded reinforcement after the reaction wall test. This procedure required demolition of the RC column after the cyclic testing. Figure 2(a) shows the accelerated corrosion procedure in the laboratory and Figure 2(b) shows the corroded column after completion of the accelerated corrosion process. The mass loss measurement of corroded reinforcement after the reaction wall test showed an average of 6.1% mass loss. Faraday's law is based on bare steel, not steel inside concrete, and therefore some differences between the theoretical Faraday's law and the experimental results are to be expected. If the corrosion rate is low there will be better agreement between the corrosion of bars inside concrete and Faraday's law. Moreover, the applied current was distributed between the longitudinal and transverse bars. Therefore, Faraday's law predicts the total mass loss of steel in an electrical circuit and this will differ from the mass loss of individual bars. A detailed discussion on this subject is available elsewhere (Kashani *et al.*, 2013a).

A lateral cyclic load was applied by means of a 50 kN actuator. Lateral deflection over the height of the column, rotation at the base and strains were measured using external

displacement transducers. The reaction wall test set up is shown in Figure 2(c). No axial force was applied and the experiment was conducted under displacement control system. A two-cycle reversed symmetrical displacement history was used. The actuator was set to displacement control with a constant displacement rate of 2 mm/s.

2.3 Experimental results and discussion

The force–lateral displacement response of the corroded column specimen is shown in Figure 3. Figure 4 shows the development of flexural damage as the drift level increased during the test. The first flexural cracks appeared at a displacement of 15 mm, corresponding to 0.6% drift. The maximum measured positive load was 20.3 kN at 3% drift (75 mm) and the maximum measured negative load was 20 kN at 3% drift (75 mm).

Horizontal cracks appeared during cycles between 0.6% and 5.0% drift, growing in number and extension, and were located at the column base and up to a height about 700 mm from the top of the foundation (Figure 4(a)). The crack density and widths were concentrated in the lower 250 mm of the column, which corresponds to the plastic hinge length. Some minor vertical cracks along the vertical bars were observed. These cracks were initially caused by the corrosion and opened up during the cyclic test. The corrosion level in this experiment was only moderate otherwise these cracks could have opened up more

Figure 2. Corroded column specimens: (a) accelerated corrosion process; (b) corroded column after accelerated corrosion procedure; (c) reaction wall test setup in the laboratory

Figure 3. Cyclic force–displacement response of tested column

significantly, as observed by other researchers (Ma *et al.*, 2012; Meda *et al.*, 2014).

After the peak horizontal force a significant strength reduction and degradation was seen in the force–displacement response. The first significant strength loss was seen at 4.5% drift (112.5 mm). The measured horizontal force at this cycle was 13 kN, which was 65% of the maximum measured load. At this point the first vertical bar fractured in tension. Figure 4(d) shows severe crack opening at 4.5% drift, which was due to the fracturing of a corner bar. Figure 4(e) shows the fracture of a vertical bar in tension. This type of failure in RC components under cyclic loading is due to low-cycle high-amplitude fatigue degradation (Kashani *et al.*, 2015b).

Following the fracture of the first vertical bar, a significant reduction was seen in the force–displacement response of the column. This was then followed by a sharp degradation in the force–displacement response. The first bar fracture was located in the corner of the column; this resulted in a significant loss of strength and permanent damage that caused the column to tilt sideways. As a result, the column only sustained two cycles after fracture of the first vertical bar and the test was therefore stopped at this point. It should be noted that the column was well confined and therefore buckling of the vertical bars was not seen and the failure was governed by the fracture of bars in tension.

3. Computational modelling for non-linear response prediction of corroded RC columns

In recent years, for non-linear analysis of RC structures subject to seismic loading, much attention has been given to the development of the fibre element technique (Spacone *et al.*, 1996a,

1996b). In this method the member cross-section is discretised into a number of steel and concrete fibres at section level. The material non-linearity is then considered through uniaxial constitutive material models of steel (tension and compression) and concrete (confined core concrete and unconfined cover concrete). Given that the stiffness of RC beam–column elements (frame elements) varies with the loading, the element response is greatly influenced by the moment–curvature (M – κ) response of the cross-section. Therefore, the element stiffness matrix in the fibre element technique is estimated based on the M – κ response of element cross-sections.

3.1 Computation of moment–curvature for a beam–column section

The theoretical force–moment–curvature (F – M – κ) relationship is obtained as follows. The main assumptions here are that plane sections remain plane and the interaction effects of shear stress and direct stress are assumed to be negligible (Euler–Bernoulli beam theory). A compression-positive sign convention is used here. Figure 5 shows a schematic overview of the F – M – κ relationship.

The linear strain distribution is given by

$$\varepsilon(y) = y\kappa_z + \varepsilon_0$$

$$1. \quad \varepsilon_0 = \left(\frac{\varepsilon_2 + \varepsilon_1}{2} \right)$$

$$\kappa_z = \frac{\varepsilon_2 - \varepsilon_1}{d}$$

The direct stress for steel is $\sigma_s = f_s(\varepsilon)$, where the function f_s is defined by the new uniaxial material model developed by Kashani *et al.* (2015a). The direct stress for concrete (confined core concrete and unconfined cover concrete) is $\sigma_c = f_c(\varepsilon)$, where the function $f_c(\varepsilon)$ is defined by Park *et al.* (1982).

By considering equilibrium between internal and external actions

$$N = \int \sigma dA \quad \text{hence}$$

$$N = \underbrace{\int \sigma_c b dy}_{\text{Concrete}} + \underbrace{\sum \sigma_s \Delta A_s}_{\text{Steel bars}} - \underbrace{\sum \sigma_c \Delta A_s}_{\text{voids}}$$

$$2. \quad S_N \equiv \left\{ \int \sigma_c b dy + \sum (\sigma_s - \sigma_c) \Delta A_s - N \right\} = 0$$

Figure 4. Development of flexural damage in the corroded column: (a) 0.6% drift; (b) 1.0% drift; (c) 3.0% drift; (d) 4.5% drift; (e) close-up view of corroded bar fracture in tension

$$M_z = \int y \sigma dA \quad \text{hence}$$

$$M_z = \underbrace{\int y \sigma_c b dy}_{\text{Concrete}} + \underbrace{\sum y \sigma_s \Delta A_s}_{\text{Steel bars}} - \underbrace{\sum y \sigma_c \Delta A_s}_{\text{Voids}}$$

$$3. \quad S_{M_z} \equiv \left\{ \int y \sigma_c b dy + \sum y (\sigma_s - \sigma_c) \Delta A_s - M_z \right\} = 0$$

The concrete stress integrals $\int \sigma_c b dy$ and $\int y \sigma_c b dy$ can be evaluated numerically using the trapezium rule given a specific

Figure 5. Schematic illustration of the M – κ relationship of RC sections

strain profile. Equations 2 and 3 represent two simultaneous non-linear equations in terms of the parameters

- strain ε_0
- curvature κ_z
- applied axial force N
- applied moment M_z
- location and size of steel bars
- stress–strain table for steel
- size and geometry of the concrete section, including possible variations in width b
- stress–strain table for concrete (including tensile strains).

Given sectional and reinforcement details including material stress–strain tables, the action N and the curvature κ_z , Equations 2 and 3 can be solved for ε_0 and M_z . A one-dimensional (1D) iterative scheme using the Newton–Raphson algorithm is used to solve this system of equations. Once the solution strain profile is determined, the flexural rigidity and axial rigidity can be calculated

$$4a. \quad EI_z = \frac{M_z}{\kappa_z}$$

$$4b. \quad EA = \int E \, dA \quad \text{hence} \\ EA = \underbrace{\int E_c b \, dy}_{\text{Concrete}} + \underbrace{\sum E_s \Delta A_s}_{\text{Steel bars}} - \underbrace{\sum E_c \Delta A_s}_{\text{Voids}}$$

The moment–curvature relationship can then be derived by executing the above algorithm for a range of curvature values. Allowance can also be made for buckling of the compression

bars (Kashani *et al.*, 2014). The effective length (L/D) used in the buckling model was taken as the ratio of tie spacing (L) to vertical bar diameter (D). Thus the reinforcing bars are allowed to buckle when the strain in the concrete is great enough to cause spalling and when the force levels in the bar exceed the critical buckling load. If no allowance is made for this effect then the theoretical moment–curvature relationship shows strain hardening.

3.2 Non-linear finite-element model using beam–column element

The typical displacement-based beam–column element using Hermitian shape functions is used for the column model (Bathe, 1996). To account for second-order effects due to geometrical non-linearity, the geometric stiffness matrix is added to the material stiffness matrix (McGuire *et al.*, 2000). A rotational spring model is used to model the slippage of the reinforcement at the interface of the column and foundation. Figure 6 shows a schematic overview of the proposed finite-element model; in the figure, w is the column width.

3.3 General solution procedure to account for material non-linearity

The element stiffness matrices are assembled in the standard way; that is, local to global transformation, assemblage of a global stiffness matrix, introduction of support restraints, production of action vector due to the applied actions and deformations. An incremental loading scheme is used where the applied actions and deformations are increased, incrementally, up to failure of the structure. At each increment the global system of linear equations is solved resulting in nodal displacements (in global coordinates). Element nodal actions can be calculated in the standard way from the element nodal displacements in local coordinates. By using the moment–curvature look-up table for a section the internal node moment and hence flexural rigidity can be calculated. Then, the element

Figure 6. Schematic illustration of the proposed finite-element model of RC column

stiffness matrices are updated for the next increment. A Newton–Raphson convergence procedure was not used here as the increments in displacement were kept small. Details of incremental size verification and mesh sensitivity are available in the literature (Cox, 2001; Noor *et al.*, 2000). In this study, ten elements were used to model the first 800 mm of the column height immediately above the foundation and five elements were used to model the rest of the column to the top. It should be noted that Figure 6 is only indicative and does not represent the actual number of elements used in the analysis.

4. Modelling the impact of corrosion on reinforcing steel and damaged concrete

4.1 Effect of corrosion on the non-linear stress–strain behaviour of reinforcing bars

The results of the tension tests showed that corrosion levels up to about 15% do not have a significant effect on the stress–strain curves. However, once the corrosion level is greater than 15% a significant drop occurs in the plastic deformation capacity and the residual capacity of the corroded bars. This is similar to the results from previous studies that used similar

reinforcement; further details and discussion are available elsewhere (Du *et al.*, 2005a, 2005b; Kashani *et al.*, 2013a).

To account for the limited strength and ductility capacity of corroded reinforcing bars, the stress–strain curve for the reinforcement is modified by changing the yield stress and fracture strain. This empirical change in yield stress and fracture strain (based on experimental data) is described by

$$5. \quad \sigma'_y = \sigma_y(1 - \beta_s\psi)$$

$$6. \quad \varepsilon'_u = \varepsilon_u(1 - \beta_e\psi)$$

where σ'_y is the yield stress of a corroded bar in tension, σ_y is the corresponding yield stress of the uncorroded bar and ψ is the percentage mass loss due to corrosion. The value of β_s is 0.005 and β_e is 0.05, as reported by Du *et al.* (2005a, 2005b). β_s and β_e are empirical coefficients, known as pitting coefficients, which account for the influence of pitting on the premature fracture and reduced capacity of corroded reinforcing bars.

4.2 Effect of corrosion on inelastic buckling of corroded bars

The empirical equations developed by Kashani *et al.* (2013a) were used to modify the compression response of corroded reinforcement. The effect of corrosion on compressive yield strength (σ'_{yc}) is defined using the empirical Equation 7, which was calibrated based on the observed experimental results

$$7. \quad \sigma'_{yc} = \begin{cases} \sigma_y(1 - 0.005\psi) & \text{for } L/D \leq 6 \\ \sigma_y(1 - 0.0065\psi) & \text{for } 6 < L/D < 10 \\ \sigma_y(1 - 0.0125\psi) & \text{for } L/D \geq 10 \end{cases}$$

Details of the experimental results and development of the above empirical equations are given by Kashani *et al.* (2013a).

4.3 The new non-linear uniaxial material model for reinforcing bars

Kashani *et al.* (2015a) developed a new uniaxial material model for reinforcing bars. The new material model accounts for the influence of corrosion damage, inelastic buckling and low-cycle fatigue degradation. The material parameters were calibrated based on experimental and numerical simulation data of uncorroded and corroded bars.

The basic tension envelope is that proposed by Balan *et al.* (1998), which employs a continuous function that provides a

smooth transition from linear elastic to the strain-hardening region. This improves numerical stability during the computational process and was therefore used to define the tension envelope

$$8. \quad \sigma = \sigma_y \frac{(1 - \mu)}{2} \left[1 + \frac{(1 + \mu) \varepsilon}{(1 - \mu) \varepsilon_y} - \sqrt{\left(\frac{\varepsilon}{\varepsilon_y} \right)^2 + \delta} \right]$$

where $\mu = E_h/E_s$ is the hardening ratio with E_s and E_h equal to the elastic modulus and hardening modulus for the steel, σ_y is the yield stress, ε is the current strain, ε_y is the yield strain and δ is a shape parameter. Equation 8 represents a hyperbola with two asymptotes, one with slope E_s and one with slope E_h . The shape parameter δ defines the curvature radius of the transition between the linear elastic and hardening regions of the curve. Further details of this model are available in Balan *et al.* (1998).

The basic compression envelope of the model employs an exponential function to describe the post-yield buckling response of the reinforcing bars. This approach has been used previously by others to model the inelastic buckling behaviour of concentric steel bracing (Hill *et al.*, 1989; Thai and Kim, 2011); here the post-buckling curve is defined in Equation 9

$$9. \quad \sigma = \begin{cases} E_s \varepsilon & : \varepsilon \leq \varepsilon_y \\ \sigma^* + (\sigma_y - \sigma^*) \exp[-(\rho_1 + \rho_2 \sqrt{\varepsilon_p})(\varepsilon_p)] & : \varepsilon > \varepsilon_y \end{cases}$$

for $8 \leq L/D \leq 30$

where ρ_1 is the initial tangent of the post-buckling response curve, ρ_2 is the rate of change of the tangent, ε is the current strain, $\varepsilon_p = \varepsilon - \varepsilon_y$ is the plastic strain, σ^* is the asymptotic lower stress limit of the post-buckling curve and all other variables are as previously defined. The parameters ρ_1 , ρ_2 and σ^* are defined by the yield strength and the geometrical slenderness ratio of the reinforcing steel, as defined by

$$10. \quad \rho_1(\lambda_p) = 4.572\lambda_p - 74.43$$

$$11. \quad \rho_2(\lambda_p) = 318.40 \exp(-0.071\lambda_p)$$

$$12. \quad \sigma^* = 3.75 \frac{\sigma_y}{L/D}$$

$$13. \quad \lambda_p = \sqrt{\frac{\sigma_y}{100}} \frac{L}{D}$$

where σ_y is in units of MPa.

Further discussion and detailed derivations of the above equations are presented by Kashani *et al.* (2015a) and Dhakal and Maekawa (2002). The yield and buckling strength and ultimate strain of corroded reinforcement in Equations 8–13 are then modified using the empirical formulas described in Sections 4.1 and 4.2.

4.4 Modelling the impact of corrosion on geometrical properties of corroded bars

Kashani *et al.* (2013c) conducted 3D optical measurements of corroded bars to explore the spatial variability of the corrosion pattern. They found that the geometrical properties of corroded bars can be modelled using a log-normal distribution. In this study, the mean values of the log-normal distribution models are used to account for the effect of pitting corrosion on the geometrical properties of corroded bars.

Equation 14 can be used to calculate the average reduced cross-section area of reinforcement considering a linear reduction in area as a function of percentage mass loss ψ

$$14. \quad A_{ave} = A_0(1 - 0.01\psi)$$

where A_{ave} is the average reduced cross-section area of corroded reinforcement and A_0 is the corresponding original uncorroded cross-section area.

Once the average reduced cross-section area is calculated, the cross-section area considering the pitting effect (A') can be calculated using

$$15. \quad A' = \gamma A_{ave}$$

where γ is the mean value of the area pitting coefficient, which is derived by assuming a log-normal distribution (further details are available in Kashani *et al.* (2013c)).

Kashani *et al.* (2013c) found that the irregular cross-sectional shape of corroded bars results in rotation of the principal axis. Therefore, in probabilistic models, they considered the minimum principal second moment of area. The minimum second moment of area of the corroded bars (I'_{min}) can be calculated by introducing a pitting coefficient for second moment of area, as defined by

$$16. \quad I'_{min} = KI_0$$

where K is the mean value of the pitting coefficient of the minimum second moment of area of corroded bars considering a log-normal distribution and I_0 is the second moment of area of the original uncorroded bar.

The mean values of the pitting coefficients (γ and K) can be calculated using

$$17. \quad M_{(\gamma \text{ or } K)} = \exp\left(\mu + \frac{\sigma^2}{2}\right)$$

where μ and σ are defined as

$$18. \quad \mu = a\psi^b$$

$$19. \quad \sigma = c\psi^d$$

in which a , b , c and d are coefficients; further details are available elsewhere (Kashani *et al.*, 2013c).

4.5 Modelling corrosion-induced cracked cover concrete

The response of cracked concrete in compression is described in detail by Vecchio and Collins (1986) and is known as compression field theory (CFT). Based on CFT, the compressive strength of cracked concrete in compression depends on the magnitude of the average tensile strain in the transverse direction, which causes longitudinal microcracks. A similar theory applies for the corrosion-induced cracking of cover concrete that is in compression. Coronelli and Gambarova (2004) employed this method in a non-linear finite-element analysis of corrosion-damaged RC beams. Equation 20 can be used to modify the compressive strength of cover concrete

$$20. \quad \sigma'_c = \frac{\sigma_c}{1 + \eta \varepsilon_1 / \varepsilon_{co}}$$

in which η is a coefficient related to bar roughness and diameter (for medium-diameter ribbed bars $\eta = 0.1$), ε_{co} is the strain at the peak compressive stress (σ_c) and ε_1 is the average (smeared) tensile strain in the cracked concrete at right angles to the direction of the applied compression (see Coronelli and Gambarova (2004) for further details).

4.6 Modelling corrosion-damaged confined concrete

Reinforced concrete bridge piers exhibit inelastic response when they are subjected to large lateral forces during major earthquakes. It is well known that the confinement associated with hoop reinforcement will increase the ductility and energy absorption capacity of RC bridge piers. However, the corrosion of horizontal tie reinforcement can change the behaviour of confined concrete under high compression loads. Here, the effect of corrosion on confined concrete is considered by reducing the volumetric ratio and yield strength of the confinement

reinforcement as a function of steel mass loss due to corrosion. The influence of corrosion on reduced ductility is also considered by limiting the maximum crushing strain in the confined concrete as a function of reduced ductility of hoop reinforcement by modifying the empirical equation developed by Scott *et al.* (1982).

Scott *et al.* (1982) defined the maximum crushing strain of confined concrete by the fracture of the first horizontal tie/spiral reinforcement. The model proposed by Scott *et al.* (1982) is by

$$21. \quad \varepsilon_{cc} = 0.004 + 1.4 \left(\frac{\rho_{sc} \sigma_{ytie} \varepsilon_{utie}}{\sigma_c} \right)$$

where ε_{utie} is the fracture strain of the tie/spiral reinforcement, σ_{ytie} is the yield strength of horizontal tie reinforcement and ρ_{sc} is the volumetric ratio of confinement reinforcement (i.e. horizontal tie reinforcement). The yield strength, fracture strain and volumetric ratio of horizontal ties reinforcement can be modified using the empirical models described in Sections 4.1, 4.2 and 4.4 of this paper.

4.7 Modelling bond-slip behaviour of column–foundation interface

In the seismic design of RC structures and bridges, plastic hinges are formed at the column/beam ends. This will induce a substantial strain penetration along the longitudinal bars into the joint, which eventually results in slippage of the longitudinal bars. This phenomenon was observed in the current column experiment and by other researchers (Lehman and Moehle, 2000). Lowes and Altoontash (2003) adopted a bar-slip model for the end slip of longitudinal reinforcement in beam–column joints.

Corrosion affects the reinforcing steel near the surface of the concrete due to diffusion of chloride ions from the surface and/or carbonation of the cover concrete. In bridge piers, the vertical reinforcement bars are anchored to the foundation well below the foundation surface. Therefore, the vertical reinforcement does not corrode at this depth and the bar-slip behaviour of the bars at the anchorage zone remains the same as in the uncorroded column. This has been observed by other researchers experimentally.

It should be pointed out that corrosion does affect the bond strength of corroded vertical reinforcement above the foundation level (internal bond-slip within the column itself). However, based on the observed experimental results, the reduced bond strength does not govern the failure of columns. Meda *et al.* (2014) and Ma *et al.* (2012) reported that the failure of corroded columns and beams under cyclic loading is

mainly governed by the fracture of bars in tension due to low-cycle fatigue and buckling of bars and crushing of confined concrete in compression.

Therefore, the bond-slip of corroded vertical bars within the element is not considered in this research. Further details of the development of the bond-slip model are available in Kashani (2014).

5. Validation and discussion of computational results

5.1 Monotonic pushover analysis result

It is assumed that the reinforcing bars (vertical and tie reinforcement) have the same percentage mass loss as measured. Given that the column is very well confined, the buckling length of the vertical bars is taken to be the same as the spacing of the tie reinforcement ($L/D = 5$). This assumption was proved to be correct by the failure mode observed in the experiment (as shown in Figure 4). The procedure explained in Section 3 together with uniaxial material models described in Section 4 were used in the non-linear pushover analysis. Figure 7 shows the results of the monotonic pushover analysis of the proposed column and a comparison with the experimental results. The numerical results show that considering the bond-slip of the base of the column does not have a significant impact on the prediction of the maximum strength of column. However, the bond-slip does influence the plastic rotation capacity (i.e. ductility of the column).

The numerical results indicate that failure starts with cracking of the cover concrete followed by fracture of the vertical reinforcement in tension. This is in good agreement with the

Figure 7. Comparison of the computational response and observed experimental response

observed experimental results. The numerical analysis and the experimental results demonstrate the importance of modelling the influence of corrosion on both steel and damaged concrete (through loss of confining tie reinforcement). Corrosion damage to the confined concrete results in a rapid reduction in strength and ductility of the corroded column under cyclic loading. It should be noted that the computational model developed in this paper is valuable for the prediction of the capacity of corroded columns, but it does not account for cyclic degradation and the low-cycle fatigue failure of vertical reinforcing bars. Further discussion about cyclic degradation and low-cycle fatigue of corroded columns is available elsewhere (Kashani, 2014; Kashani *et al.*, 2015a).

The numerical model showed that, in the absence of axial force and inelastic buckling, the failure mode is governed by the fracture of vertical bars in tension. However, this may not be valid for columns with axial force with buckling. Therefore, the validated numerical model was used to explore the effect of axial force, inelastic buckling and corrosion damage on the residual capacity of RC column sections, as reported in Section 5.2.

5.2 Impact of corrosion on the non-linear response and capacity of RC sections

The proposed numerical model was validated against a benchmark experimental test. To demonstrate the influence of corrosion on capacity reduction and the inelastic response of RC sections, a series of moment-curvature ($M-\kappa$) analyses was conducted on a hypothetical column with the same cross-section properties (dimension, reinforcement etc.) as the tested column. To investigate the combined impact of corrosion and bar buckling on inelastic section response, $L/D = 10$ is assumed in the analyses. The results of these analyses are shown in Figure 8. To show the significance of buckling, $M-\kappa$ analyses were conducted for uncorroded sections with and without the bar buckling effect. The $M-\kappa$ analyses of corroded sections are only considered with the effect of buckling.

Figure 8(a) shows that corrosion has a significant impact on the flexural rigidity and ductility of RC sections. It should be noted that the tested column had no axial force but the axial force-bending moment interaction is included in the numerical model. The impact of axial force on the non-linear section response is more severe where the inelastic buckling of vertical bars is critical. A high axial force results in spalling of cover concrete at a lower drift ratio, followed by inelastic buckling of vertical bars. Once the vertical bars buckle, they lose strength; this subsequently increases the stress in the core concrete (concrete confined within hoops). Therefore, the core concrete crushes soon after buckling. This can be seen in Figure 8(a), which compares the moment-curvature response of the hypothetical RC section with and without considering buckling.

Figure 8. Impact of corrosion on inelastic response of RC section: (a) moment–curvature response and (b) axial force–bending moment interaction response; A_g , area of gross cross section; f_{cu} , compressive strength of concrete

In the $M-\kappa$ analysis, if the strain in the extreme fibre is limited to a fixed value (i.e. concrete crushing strain in compression) and the $M-\kappa$ analysis is repeated for a range of axial forces, an axial force–bending moment ($N-M$) interaction diagram can be generated. Figure 8(b) shows the $N-M$ interaction diagram of the same section for varying corrosion levels. It is evident from the figure that corrosion has a significant impact on the capacity of RC sections.

The results of this study show that material degradation has a significant impact on the non-linear response of RC structures

at section and component level. This will subsequently affect the system response (i.e. the response of a whole bridge subject to increased live load over the service life and/or earthquake loading). Therefore, considering only a uniform area loss of reinforcing bars in structural evaluation of corrosion-damaged bridges in both seismic and non-seismic regions is not sufficiently accurate. The assessment methodology and guidelines developed here significantly improve on previous methods. Moreover, the computer code is relatively simple and can be implemented in any standard section analysis software or engineering spreadsheets used in industry. However, there is still a

need for further experimental studies on corroded RC components for further validation and calibration of the model under various loading protocols and corrosion scenarios. Nevertheless, this method is currently the only available and reliable computational platform for prediction of the residual capacity of corrosion-damaged RC columns/bridge piers.

6. Conclusions

The outcome of this study resulted in a significant improvement to existing models for the structural evaluation and seismic assessment of corrosion-damaged RC bridge piers. The new uniaxial material models have enhanced the accuracy of non-linear beam-column models for predicting the non-linear response of RC sections and columns.

The main outcomes of this study can be summarised as follows.

- (a) It is inadequate to assume that corrosion only affects the main vertical reinforcement in a column. It was found that confined concrete with corroded confinement reinforcement starts crushing much faster than uncorroded undamaged concrete. This change in the failure mode cannot be predicted if the damage in core confined concrete due to corrosion of tie reinforcements is ignored.
- (b) Corrosion-induced damage to horizontal tie reinforcements results in premature buckling of the vertical reinforcement. This is in good agreement with the experimental results reported by other researchers (Ma *et al.*, 2012; Meda *et al.*, 2014). The computational platform developed in this paper is capable of predicting this failure mode.
- (c) The results of numerical analyses of uncorroded and corroded RC column sections showed that inelastic buckling of vertical bars changes the failure mode of columns subject to lateral loading. In the absence of buckling, the failure mode is governed by the fracture of bars in tension. However, inelastic buckling of vertical bars results in premature crushing of the core concrete. Therefore, the failure mode is governed by the crushing of core concrete in compression. It should be noted that the level of axial force applied to the column is also important, and this is included in the proposed numerical model.
- (d) The modelling technique developed in this paper has significantly improved earlier models and can be used by other researchers and practising engineers for structural capacity assessment and evaluation of corrosion-damaged RC columns and sections.

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