Durability Considerations in Performance-Based Seismic Assessment of Deteriorated RC Bridges

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SUMMARY:
For sustainable development of the urban environment where people and nature reside together, infrastructures are required to retain their performance over the long-term. In order to provide a durable and reliable solution, functionality, safety and the service life of infrastructures must be estimated based on scientific and engineering knowledge. Corrosion of reinforcing steel is one of the primary sources of durability problems in Reinforced Concrete (RC) Bridges. This leads to reduction in the steel cross section and weakening of the bond and anchorage between the concrete and reinforcement, which directly affects the serviceability and ultimate strength of concrete elements within a structure. Currently there is a gap in our knowledge about how this long-term material deterioration over the life span of structure affects the non-linear behaviour of RC bridges subject to seismic loading. This paper outlines some of the recent developments toward this goal.

Keywords: Corrosion, Performance-Based Assessment, Durability of RC Bridges, Bridge Management System

1. INTRODUCTION

Over the past couple of decades researchers have made significant efforts to improve the general knowledge of seismic design and analysis of structures in earthquake prone regions. As a result of these efforts the Performance-Based Earthquake Engineering (PBEE) framework has been developed which is currently very well stabilised among researchers and practising engineers around the world. Based on the observed response of existing structures in recent large earthquakes, it is clear that most structures designed and constructed after 1990 have performed well. However there are a significant number of older major infrastructure artefacts that are located in an aggressive environment and are suffering from the material aging and deterioration. In order to provide a durable and reliable solution, the functionality, safety and the service life of various infrastructure artefacts must be estimated based on scientific and engineering knowledge. Therefore for existing deteriorated infrastructure a rational maintenance plan should be employed in accordance with their current condition. Climate change, sustainability issues and limited funding in the recent years means there is a need for a new bridge management system that can capture all this complexity into a single comprehensive framework. This has led researchers around the world to consider extending the existing performance based earthquake engineering framework to include the long term material performance.

Among the different deterioration mechanisms, corrosion of reinforcing steel is the most common reason for the premature deterioration of RC structures in a chloride laden environment. This leads to loss of the steel cross section and weakening of the bond and anchorage between concrete and reinforcement, which directly affects the serviceability and ultimate strength of concrete elements within a structure. This will increase the probability of failure of deteriorated structures in big earthquakes. It can also result in significant permanent damage in smaller earthquakes which will increase the Whole Life Cycle Cost (WLCC) of the structure. In the following sections a solution procedure to this complex problem is discussed.

2. BRIDGE MAINTENANCE MANAGEMENT VERSUS PERFORMANCE-BASED SEISMIC ANALYSIS OF BRIDGES

Bridge maintenance management is a challenging task that involves the identification of optimal prioritisation of maintenance and rehabilitation for bridge structures and the determination of optimal rehabilitation strategy for each structure of a given bridge or a network of hundreds or thousands of bridges. This optimisation should consider several objectives that may be of a conflicting nature, such as minimisation of failure risk and minimisation of rehabilitation costs over the life cycle of the structure. To achieve this goal there is a need to develop and integrate reliable and effective decision support models that include: Condition assessment models; Deterioration prediction models, Risk assessment models, and Maintenance optimisation models as shown in Figure 1 (Z.Lounis 2003).

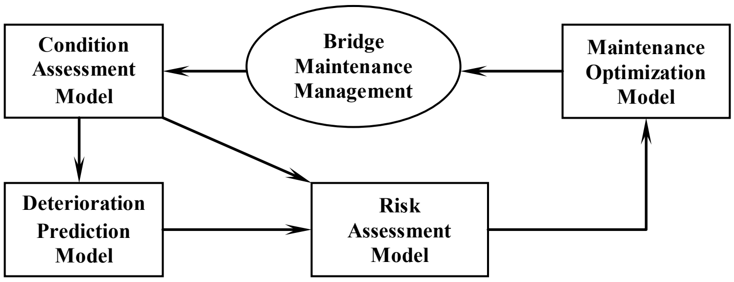

Figure 1 Decision models for bridge management

On the other hand if the bridge network is located in an earthquake prone region the seismic risk assessment must be included in the system. Unlike buildings where collapse hazard to occupants, repair costs, and loss of functionality are all significant considerations, the overriding performance metric for bridges is post-earthquake functionality, i.e., the reduced capacity of a bridge coupled with the required time to restore the bridge to full functionality. In other words, the bridge fragility relationships are the key components to the performance assessment of the overall highway network (Moehle and Dierlein 2004).

In some recent research Flint and Billington (2011) developed a probabilistic framework for Performance Based Durability Engineering (PBDE), Alipour et. al. (2011) developed a multi hazard framework that evaluates life-cycle performance and cost of highway bridges in earthquake prone regions. However, in order to have an efficient framework the structural model must be based on the realistic modelling of time dependent constitutive material behaviour. Currently, there is not enough data available to indicate how corrosion might change the constitutive behaviour of damaged materials. Towards this, similar framework is being developed in this research with emphasise on the effect corrosion on nonlinear seismic response and capacity loss estimation. The new framework accounts for the combined effect of seismic hazard and ongoing deterioration of existing RC bridges with enhanced time-dependent material models. This has been simplified and summarised in Figure 2.

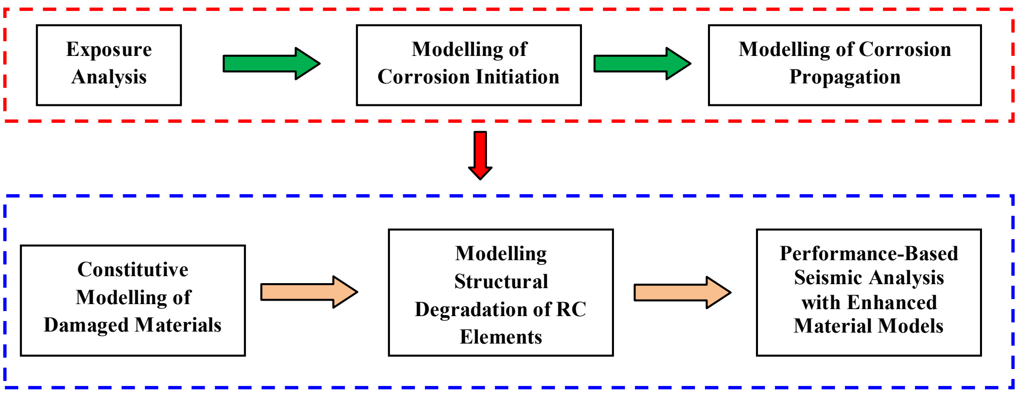

Figure 2 Simplified Procedure of the Durability-Based Seismic Performance Assessment (DBSPA)

3. MODELLING CHLORIDE INDUCED CORROSION

Numerous models have been proposed for predicting the onset and rate of corrosion of steel reinforcement in concrete exposed to chlorides (P.B. Bamforth 2004). In general these models assume a two-stage process, defined as initiation and propagation periods as first proposed by Tuutti (P.B. Bamforth 2004).

The rate of ingress is often approximated to Fick's Law of diffusion with some further complications here. The initial mechanism appears to be suction, especially when the surface is dry, that is, capillary action. Salt water is rapidly absorbed by dry concrete. There are then some capillary movements of the salt laden water through the pores followed by 'true' diffusion (J.P. Broomfield, 2007).

A concentration gradient is considered as the common driving force. Given the fact that concrete is a heterogeneous and aging material, temporal and spatial variability is associated with the diffusion coefficient. Since in the field, chloride ingress occurs under transient conditions, Fick's second law of diffusion can be used to describe the time variation of chloride concentration for one-dimensional flow, as follows (Kashani 2006):

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}[D\frac{\partial C}{\partial x}] \qquad \textbf{(3.1)}$$

where, C is chloride concentration at a distance x from the surface after the time t, D is diffusion coefficient. Under the assumption of a constant diffusion coefficient, and boundary condition specified as $C = C_s$ and the initial condition specified as $C = 0$ for $x>0$, $t = 0$, Crank's solution to the Eq. (3.1) yields (J. Crank, 1975):

$$C(x,t) = C_s[1 - erf(\frac{x}{2\sqrt{Dt}})] \qquad \textbf{(3.2)}$$

Where, $C(x,t)$ is the chloride concentration at depth $\boldsymbol{x}$ after time t, erf is the error function, C_s is chloride surface concentration (% weight of cement), D is diffusion coefficient *(mm^2/year)*, From this, it is possible to evaluate effective diffusion coefficient value, D and the time for chloride initiated corrosion to begin by setting $C(x,t)$ equal to critical chloride threshold C_{cr} based on the following assumptions (M. Chryssanthopoulos, G. Sterritt, 2002):

i. One dimensional diffusion in semi-infinite homogeneous body is representative of the chloride ingress process in concrete structures
ii. Chloride surface concentration, Cs, is constant through time
iii. The diffusion coefficient, D is spatially constant

3.1. Modelling corrosion initiation period

With respect to the Eq. 3.2 itself, the main drawbacks that it presents is the D value which is indicative of the effect the *Cx/Cs* ratio and therefore a *Cs* should be always given when a D value is calculated. This can be deducted from Figure 2 where several chloride profiles are represented from the chloride drilling test on a real bridge. Although the amount of chlorides at rebar level is the same in profiles named A and B, the value of diffusion coefficient from profile B is lower than that of profile A and similar to those of C and D. Sometimes the *Cs* is not the highest value of the profile, but a maximum in chloride concentration is found beyond the concrete surface. In these cases, the extrapolation back to the surface gives an apparent *Cs* whose corresponding D is called Apparent Diffusion Coefficient and may lead to erroneous predictions, because the further evolution of the diffusion is provided by the maximum and not by the apparent *Cs* mathematically obtained.

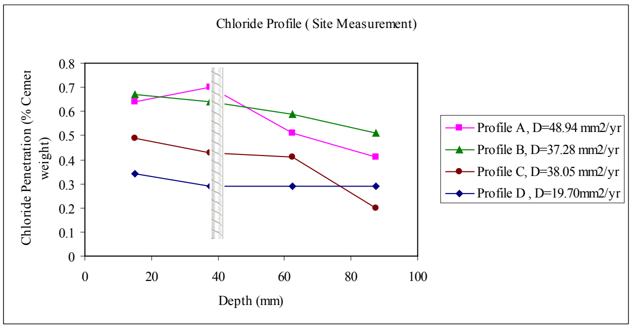

Figure 3 Chloride profile examples

Kashani and Crewe (2009) proposed a simplified method for calculation of the chloride parameters using characteristic values based on the Eurocode EN 206-1.

From the chloride profile and draw back calculation of the error function the chloride parameters *D*, *Cs* and 95% fractile values can be estimated.
Let C_{cr} to be a critical chloride corrosion threshold and x the depth of concrete (x=cover at reinforcement level), then the corrosion initiation period T_{corr} can easily be calculated from Eq. (3.3) as below:

$$T_{corr} = \frac{x^2}{4D}(1 - erf^{-1}(\frac{C_{cr}}{Cs}))^{-2} \qquad \textbf{(3.3)}$$

3.2. Modelling Time Dependent Crack Propagation Period

The following empirical expression can be used to describe the crack propagation and estimation of crack width due to corrosion (Vidal, 2004)

$$w = K(\Delta A_s - \Delta A_{s0}) \qquad (mm) \qquad \textbf{(3.4)}$$

where, ΔA_s = Steel loss of the reinforcing bar cross section in mm^2; ΔA_{s0} = Steel loss of section needed for crack initiation in mm^2; and $K = 0.0575$ which is an empirical coefficient;

To evaluate $(\Delta A_s - \Delta A_{s0})$ additional penetration of corrosion Δp, at time t after crack initiation can then be calculated as (Dimitri V. Val 2007).

$$\Delta p(t) = 0.0116 i_{corr}(t)(t - t_{cr} - T_{corr}) \quad (mm) \qquad \textbf{(3.5)}$$

where, t_{cr} is the crack initiation time. Vu and Stewart (2000) developed an equation for calculation of time-dependant corrosion rate. The corrosion current density at time t is expressed as:

$$i_{corr}(t) = 0.85 i_{corr,0}(t - T_{corr})^{-0.29} \qquad t \geq T_{corr} \qquad \textbf{(3.6)}$$

where $i_{Corr,0}$ denotes the corrosion current density at the initiation of corrosion propagation which can be written as:

$$i_{corr,0} = \frac{37.5\left(1 - \frac{w}{c}\right)^{-1.64}}{d_c} \qquad \mu A/cm^2 \qquad \textbf{(3.7)}$$

Where w/c is the water cement ratio and d_c is the cover depth, which is the distance from the surface of steel bar to the surface of the concrete structure.

and

$$\Delta A_s - \Delta A_{s0} = \pi\Delta p[(D - 2p_0 - \Delta p(t)]$$

(3.8)

where, $P_0 = \alpha W_{crit}/(\pi D\rho_{st})$, which is corrosion penetration corresponding to ΔA_{s0} and W_{crit} is the critical amount of corrosion needed to induce cracks in the cover concrete. Further detail is available in Kashani and Crewe 2009.

4. MODELLING CORROSION INDUCED MATERIAL DEGRADATION

4.1. Residual Cross Section of Corroded Reinforcement

The reduction of reinforcement cross section is function of corrosion rate which is a time dependent phenomenon. Therefore the reduced diameter D(t) of a corroding reinforcing bar at time *t* after corrosion initiation can be computed as:

$$D(t) = D_0\sqrt{1-\alpha} \tag{4.1}$$

Where, D_0 is the original uncorroded diameter of reinforcement and α is the amount of mass loss based on Faraday's law (Kashani et al. 2012).

4.2. Nonlinear Stress-Strain Behaviour of Corroded Reinforcement

A series of experimental test carried out by Kashani et al. (2012 (a) and (b)) to investigate the effect of corrosion on nonlinear stress strain behaviour of reinforcement in tension and compression. The results of tension tests show that corrosion levels up to about 15% don't have a significant effect on stress-strain curves. However, once the corrosion level is greater than 15% a significant drop occurs in plastic deformation capacity and the residual capacity of the corroded bars. This is similar to the results from previous studies which used British manufactured reinforcement (Du et. al. 2005). Figure 4 shows the observed stress-strain curves of tension tests based on average reduced areas and the relevant code requirements for ductile design (BS 4449-2009). It should be noted that a 100mm gauge was used to measure the bar extension in these tests. In many cases the rebar fracture occurred within this gauged section but because the point of fracture depends on the pitting corrosion the fracture sometimes occurred outside the gauge, as indicated in the Figure 4, which affected the strain recorded at failure.

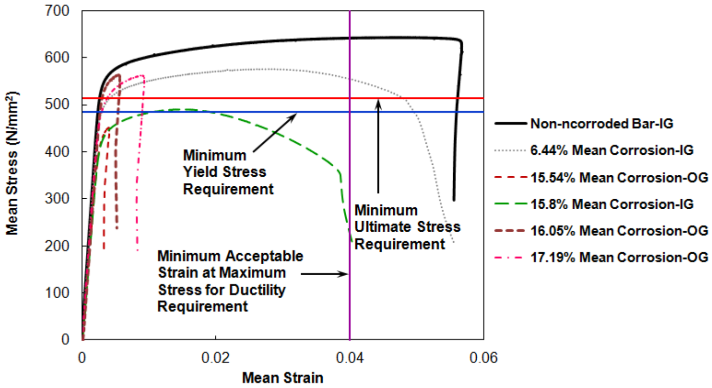

Fig. 4 Mean stress-strain curves of tension tests for the 8mm diameter bars (IG = Failure Inside the Gauge, OG = Failure Outside the Gauge)

A total of 57 monotonic compression tests were carried out on corroded bars with different effective lengths. The buckling length of bars considered in the experiment chose based on the ratio of spacing of horizontal ties (L) in the common construction of RC columns to bar diameter (D) known as L/D ratio. The L/D ratios tested in this experiment are 5, 8, 10, 15 and 20.

As expected reinforcement of the group of L/D=5 showed more stable behaviour compared to those with greater L/D ratios. Almost all the rebars were quite stable up to the yield stress and instability started after yielding which was then followed by buckling and softening behaviour. Unsymmetrical corrosion resulted an uneven reduction of cross section along the length of the corroded bar. This produced an imperfection which caused an additional stress in the bar due to load eccentricity. Therefore a corroded reinforcement yields earlier and its buckling load is reduced compared to uncorroded reinforcement. In addition, the cross sections of corroded bars are not circular any more. This unsymmetrical shape of cross section creates strong and weak axes in the reinforcement and changes the radius of gyration of corroded bars in different axes. This directly affects the theoretical buckling load of bars. As a result, different buckling collapse mechanisms were observed in each test relative to the distribution of pitting corrosion along the bar. Figure 5 shows an example of the response of 12mm diameter corroded bars in compression with slenderness ratio of L/D=15.

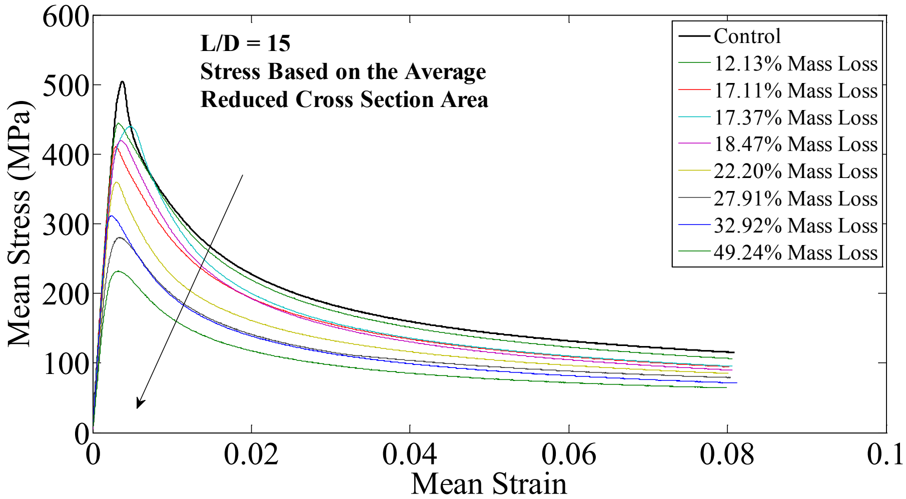

Fig. 5. Mean stress-strain curves of corroded bars in compression

4.3. Residual Capacity of Cracked Cover Concrete

The response of cracked concrete in compression is described in detail by Vecchio and Collins (1986) which 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. Similar theory applies in the corrosion induced cracking of cover concrete where it is in compression. Coronelli and Gambarova (2004) employed this method in nonlinear finite element analysis of corrosion damaged RC beams as shown in Figure 6.

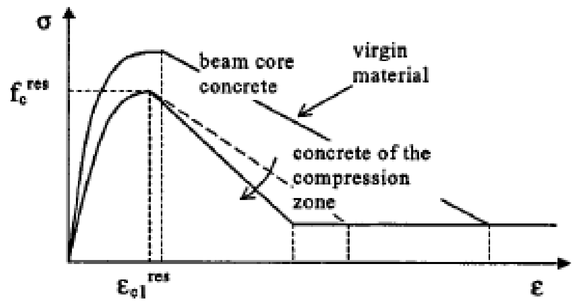

Figure 6 constitutive Model of Cracked Cover Concrete in Compression

The Eq. (4.2) can be used to modify the compressive strength of cover concrete as follows:

$$f_c' = \frac{f_c}{1 + K\dfrac{\varepsilon_1}{\varepsilon_{c_0}}} \tag{4.2}$$

where, K is the coefficient related to bar roughness and diameter (for medium-diameter ribbed bars a value K=0.1); ε_{co}= strain at the peak compressive stress f_c ; and ε_1=average (smeared) tensile strain in the cracked concrete at right angles to the direction of the applied compression. Further detail is available in Coronelli and Gambarova (2004).

5. MOMENT-CURVATURE RESPONSE OF CORROSION DAMAGED RC COLUMN SECTIONS

In recent years, for nonlinear analysis of RC structures subject to seismic loading, a lot of attention has been given to the development of the fibre element technique. In this method the member cross section is discretized into a number of steel and concrete fibres at selected integration points. The material nonlinearity is then considered through a uniaxial constitutive material model of steel (tension and compression) and concrete (confined core concrete and unconfined cover concrete). More recently, other researchers have attempted to investigate the effect of reinforcement corrosion on the behaviour and response of RC bridges subject to seismic loading through nonlinear finite element analysis using fibre-based section discretization technique (Ghosh and Padgett 2010). However, due to lack of available data and constitutive nonlinear material models of corroded reinforcing bars their analytical model couldn't represent the real behaviour of corroded structure. This is more critical in compression zones where buckling of bars and confined concrete behaviour make an important contribution to the response of the structure.

In the fibre-based section discretization technique the element response is greatly influenced by the moment-curvature response of cross section. Therefore, a moment-curvature analysis has been done for the proposed experimental specimens for both uncorroded and corroded conditions. The moment-curvature analysis is developed based on the fibre-base section discretization technique using MATLAB computer code. An unconfined concrete model is used for cover concrete and a confined concrete model is used for the core concrete (Kashani et al. 2010). The mechanical properties and cross sectional area of longitudinal reinforcement in tension have been modified using existing mathematical models available in the literature and experimental data (Kashani et al. 2012). The Dhakal-Makeawa buckling model (Dhakal-Makeawa 2002) has been modified based on the experimental tests on corroded reinforcement in compression and is included in the code to model the effect of reinforcement buckling (Kashani et al. 2012) as shown in Fig. 7. The effect of corrosion induced cracking of cover concrete in the compression zone is also considered in the analysis (Coronelli and Gambarova 2004). This analysis is the basis of a multi-mechanical nonlinear finite element code for push-over analysis of corroded bridge piers using a fibre element technique which can account for corrosion damage.

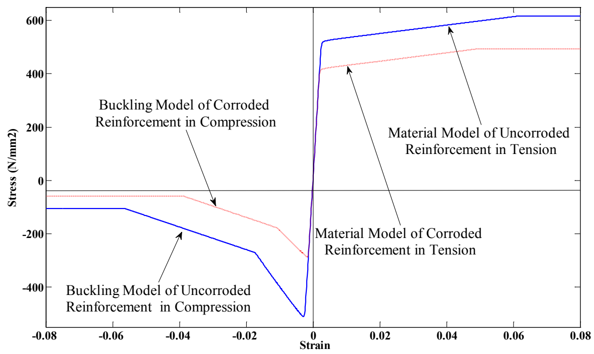

Fig. 7 Non-linear uniaxial material model of corroded reinforcing steel (L/D=12)

Figure 8 shows the analysis results. The results show that buckling has a significant effect on the moment-curvature response and plastic rotation of the corrosion damaged section. The ultimate capacity also greatly influence by the buckling and reduced tension capacity of reinforcement in tension.

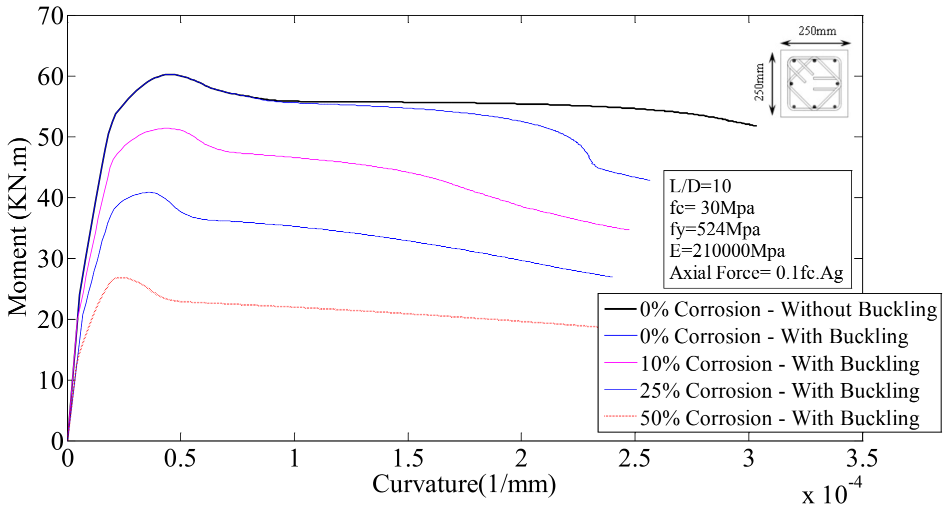

Fig. 8 Moment Curvature Response of Corrosion Damaged RC Column Section

CONCLUSION

The time dependent deterioration mechanism and modelling strategies are outlined. Furthermore a simplified framework is developed to model the time dependent capacity loss of RC bridge piers. The new modelling strategy accounts for time dependent material behaviour owing to the level of corrosion damage. Furthermore, based on the results of the recent experimental studies on the cyclic behaviour of corroded beams and columns (Ou et al. 2011, and Ma et al. 2012) there are evidence that the buckling and/or fracture of corroded bars had a significant effect on the global response, plastic rotation capacity and plastic hinging mechanisms of the corroded RC elements. Similar behaviour is observed from the moment-curvature analysis of corrosion damaged RC section. The new material models have improved the prediction of analytical model. As a result in the seismic assessment and evaluation of existing corroded structures consideration needs to be given to the buckling of bars even if the structure is originally designed to have sufficient level of confinement and anti-buckling reinforcement.

Further work required for better understanding of the spatial variability of corrosion in the corroded elements. Significant research required to understand the global buckling and overall stability of vertical reinforcement in corrosion damaged in RC columns.

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