Experimental Investigation of Nonlinear Cyclic Behaviour of Circular Concrete Bridge Piers with Pitting Corrosion

Mohammed M. Kashani

Associate Professor

Email: *mehdi.kashani@soton.ac.uk*

Address: Room 4019, Building 178,

University of Southampton

Boldrewood Innovation Campus, Burgess Road, Southampton, SO16 7QF

United Kingdom

Hamish Moodley

PhD Candidate

Email: H.T.M.Moodley@soton.ac.uk

Address: Building 178,

University of Southampton

Boldrewood Innovation Campus, Burgess Road, Southampton, SO16 7QF

United Kingdom

Hammed O. Aminulai

PhD Candidate

Email: *h.o.aminulai@soton.ac.uk*

Address: Building 178,

University of Southampton

Boldrewood Innovation Campus, Burgess Road, Southampton, SO16 7QF

United Kingdom

Sheida Afshan

Associate Professor

Email: *S.Afshan@soton.ac.uk*

Address: Room 4021, Building 178,

University of Southampton

Boldrewood Innovation Campus, Burgess Road, Southampton, SO16 7QF

Duncan Crump

Experimental Officer

Email: D.A.Crump@soton.ac.uk

Address: Large-Scale Structural Testing Laboratory (LSTL), Building 178,

University of Southampton

Boldrewood Innovation Campus, Burgess Road, Southampton, SO16 7QF

United Kingdom

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Abstract

In this study three RC columns with and without corrosion and different reinforcement details are tested under lateral cyclic loading. One of these columns is well-confined to represent the modern RC bridge piers designed according to the current seismic design codes, and the second column is of the same detail with corrosion damage. The third column is a lightly-confined corroded column to represent ageing RC bridge piers that are not designed to the current seismic design codes. The experimental results showed that corrosion has a more significant impact on the ductility loss than the strength loss of the tested corroded columns. Furthermore, although the uncorroded column was designed in accordance with the current seismic design code, severe inelastic buckling of the vertical bars was still observed during the cyclic tests.

Keywords: bridge pier, reinforced concrete, corrosion, inelastic buckling, seismic performance, nonlinear behaviour

Introduction

There are many transport infrastructure around the world, which are subject to material ageing. Deterioration of concrete bridges, which are the most critical nodes in any transport infrastructure networks is recognised as one of the major challenges facing the bridge engineering community. Reinforced Concrete (RC) structures are vulnerable to deterioration effects caused by chloride-induced corrosion (from de-icing salts and seawater) and, to a lesser extent, by carbonation (Gaal, 2004). Chloride-induced corrosion of reinforcing steel is the most significant environmental threat affecting the performance of ageing RC bridges and structures in the UK and worldwide (Broomfield, 2003; ICE, 2016). Severe corrosion and insufficient reinforcement detail have resulted in several catastrophic failures worldwide (e.g. Monardi bridge collapse in Italy, De la Concorde bridge collapse in Canada, Ynys-y-Gwas bridge collapse in UK), or severe disruption in traffic flow due to bridge closure (e.g. Hammersmith Flyover in London; carrying 100,000 vehicles per day). England's strategic and local road networks have a net worth of £344 billion (Barker et. al., 2014). Corrosion damage to RC bridges is estimated to cost about £1 billion/year in England and Wales (Barker et. al., 2014), which represents about 10% of the total UK bridge inventory. In the US, the estimated direct cost to repair ageing infrastructure is over $200 billion in total (Angst, 2018; ASCE, 2021).

A large portion of ageing corroded RC bridges are located in high seismicity regions. Therefore, several researchers (Biondini et. al., 2015; Camnasio, 2013; Dizaj et al. 2018a; Dizaj et al. 2018b; Dizaj et al. 2023; Ghosh and Padgett, 2010; Li et. al., 2015; Rao et. al. 2017) have investigated the impact of corrosion on the seismic fragility and life cycle cost of RC structures using simplified models. The focus of these studies was mainly on numerical modelling and probabilistic study on the effect of corrosion on seismic fragility of ageing bridges. They concluded that deterioration of bridges due to reinforcement corrosion has a significant negative influence on the structural vulnerability of RC bridges and increases the whole life cycle cost of such bridges significantly. Others (Cho, 2009; Du et. al., 2005a; Di Carlo et. al., 2023l Du et. al., 2005b; Imperatore et. al., 2017; Lee and Cho, 2009; Kashani, 2017; Kashani et. al., 2015; Kashani et. al., 2013; Kashani et. al., 2015) investigated the impact of corrosion on residual capacity of reinforcing bars subject to monotonic tension and compression and cyclic loading including the effects of inelastic buckling and low cycle fatigue. The outcomes of these studies provided modelling approach to simulate the uniaxial material behaviour of corroded bars under different loading scenarios. Limited experimental studies have been conducted to investigate the impact of corrosion on the nonlinear behaviour of RC components (Ge et. al., 2020; Lee et. al., 2003; Liu et. al., 2017; Meda et. al., 2014; Rajput and Sharma, 2018; Rinaldi et. al., 2022; Yuan et. al., 2017; Yang et. al., 2016). The focus of these studies was mainly on corroded RC beams or rectangular/square RC columns. The outcomes of previous experimental observations confirmed that corrosion has a significant negative impact on residual strength, multiple failure modes (e.g. flexure, or shear-flexure failure), and overall ductility of RC components (Ge et. al., 2020; Lee et. al., 2003; Liu et. al., 2017; Meda et. al., 2014; Rajput and Sharma, 2018; Rinaldi et. al., 2022; Yuan et. al., 2017; Yang et. al., 2016).

Kashani et al. (2019) reports the results of a literature survey on the available experimental data of corroded RC components. The review results revealed that most of the previous research on experimental testing of corroded RC structural components have been mainly focused on beams under monotonic and cyclic loading (flexure and shear) and rectangular/square columns subject to lateral cyclic loading. There is very limited reliable experimental data currently available in the literature to investigate the nonlinear cyclic behaviour of circular corroded RC columns (Aquino and Hawkins, 2007; Ma et. al., 2012; Yuan et. al., 2017). Circular columns are very common in bridge pier construction, and their failure mechanism is very different from rectangular/square columns owing to the difference in their geometry. Therefore, there is an urgent need for experimental investigation of nonlinear behaviour of corroded circular columns subject to cyclic loading.

Research novelty and contribution

As discussed in the previous section of this paper, there is a significant paucity of reliable experimental data in the literature on nonlinear cyclic behaviour and seismic performance of circular corroded RC bridge piers. Most of the ageing circular corroded bridge piers were designed and constructed prior to the modern seismic design codes (pre 1990s). The recently constructed circular RC bridges, which are designed and built according to the modern seismic codes, are also vulnerable to corrosion. Therefore, it is crucial to investigate the nonlinear behaviour of new and old generation of corroded circular bridge piers subject to lateral cyclic loading. There is currently no experimental data in the literature to investigate and compare the impact of corrosion on code-conforming and non-code-conforming circular RC columns. To this end, this experimental study aims to address this gap by conducting a set of benchmark experimental testing on circular corroded RC columns with different reinforcement details. The test specimens consist of a corroded and an uncorroded columns, which are designed according to Eurocode 2 (CEN, 2004) and are seismically detailed according to Eurocode 8 (CEN, 2005) to represent new bridge design. A further corroded column is designed to have the same flexural capacity as the other two columns but without seismic reinforcement detail to represent old/non-code-conforming bridge piers (pre-modern seismic design codes). The only difference in the two groups of columns is the volumetric ratio of the confinement reinforcement, which is the most important parameter in nonlinear seismic behaviour of RC bridge piers. The experimental results showed that pitting corrosion has a significant impact on the ductility and hysteretic energy dissipation capacity of RC columns, and to a lesser extent on their residual strengths.

Experimental Programme

Specimen design and properties

Three circular RC columns with 400mm diameter cross-section and 1600mm high (height above the foundation) are designed according to Eurocode 2 (CEN, 2004). The column section contained nine 16mm diameter vertical bars. Two of the columns were detailed for seismic loading according to Eurocode 8 (CEN, 2005) with the tie reinforcement spacing at 80 mm. The third column was designed according to Eurocode 2 with the same flexural capacity as the other two columns, but it was not detailed for seismic loading, with the tie reinforcement spacing at 200 mm. This column represents non-code conforming old bridge design with light confining reinforcement. The cover concrete was 30mm and the maximum aggregate size of the concrete was 10mm. Figure 1 shows the details of the column specimens, and Table 1 shows the experimental test matrix and associated concrete strength.

Table 2 and 3 summarise the mechanical properties of the steel and concrete mix used in test specimens and Figure 2 shows the nonlinear stress-strain behaviour of 8mm and 16mm diameter bars.

Accelerated corrosion procedure

The natural corrosion of RC structures on-site is a gradual process that takes several years to occur. In laboratory settings, researchers have employed various corrosion simulation methods to accelerate the deterioration of RC test specimens. Previous studies have utilised techniques such as the external current method (El Maaddawy and Soudki, 2003), pre-admixed chlorides (El Maaddawy and Soudki, 2003), and cyclic wetting and drying (Otieno et al., 2019) to expedite the corrosion process. In this study, we adopted an accelerated corrosion procedure that has been successfully employed in prior research conducted by the authors (Aminulai et al., 2023a; Aminulai et al., 2023b; Ge et al., 2020).

The detailed methodology for this accelerated corrosion procedure, utilizing external current methods and the corresponding experimental setup, can be found in the references mentioned (Aminulai et al., 2023a; Aminulai et al., 2023b; Ge et al., 2020). In brief, the approach involves establishing an electrochemical circuit using an external power source. Within this setup, the reinforcing bars function as the anode, while an external material serves as the cathode. Common cathode materials include copper, stainless steel, and regular carbon steel. An electrolyte, typically a saline solution, facilitates the flow of ionic current from the embedded reinforcement to the external cathode. In this specific experiment, stainless steel plates were utilized as the external cathode, paired with a 5% sodium chloride (NaCl) saline solution.

The accelerated corrosion procedure took eight and six weeks for columns A1 and B1, respectively. During this period, the average current applied was 5A. Figure 3 shows the corroded columns after the accelerated corrosion procedure, where some surface horizontal and vertical cracks can be observed. The vertical cracks are due to the corrosion of longitudinal/vertical reinforcing bars, and the horizontal cracks are due the corrosion of horizontal hoop/tie reinforcements.

Reaction frame test setup, instrumentation, and loading protocol

A specially designed test rig for performing lateral cyclic loading on large-scale structural components was utilised for the column tests in the Large Structures Testing Laboratory (LSTL) at the University of Southampton. Figure 4 shows the adopted test set-up, which involves a 250kN capacity MTS actuator with a 250mm stroke for applying the lateral cyclic loading. The columns were not subjected to axial load. The reaction frame and the foundation block were fixed to the laboratory's strong floor using pre-tensioned steel rods to prevent any movement during testing. Lateral displacement was applied at 1.8m at the top of the column using a displacement-controlled loading scheme, as shown in Figure 5. The lateral displacements ranged from 1.6 to 96mm, with 2 repeated cycles for each lateral deformation level as recommended by ACI 374.2R-13 (ACI, 2013). Lateral displacement in the direction away from and towards the reaction frame are assigned as positive and negative, respectively.

The measurement instrumentation utilised in the tests comprised of 5 LVDTS to measure the displacement of the column at different heights, and Digital Image Correlation (DIC) to capture the full-field strain in the plastic hinge region of the columns. DIC is a non-contact imaging technique that measures displacements and strains in structures as they deform. The process involves taking a reference image of the region of interest on the column specimen before deformation occurs, followed by continuously capturing images during deformation, and referred to as the deformed images. The deformed images are then compared with the reference image to compute displacements and strains in the region of interest. To enable image comparison, a random speckle pattern is applied to the specimen. These speckles are grouped together in DIC into subsets of at least three speckles. The deformation of each subset is used to correlate the displacements and strains of the plastic hinge region of the column. As the column is circular in geometry with a curved surface, stereo DIC with multiple cameras is employed. For these tests, four Manta G504-B cameras with two Nikon AF 50mm f/1.8D lenses and two Nikon 28mm f/2.8D lenses were used to film at a frame rate of 1 Hz throughout testing. DIC images were processed using MatchID Stereo software. Figure 6 depicts an example DIC speckle pattern. Furthermore, the cameras, settings and parameters used in DIC are presented in Table 4.

Corrosion Measurement

After structural tests, the corroded columns were carefully demolished and the mass loss of each individual vertical bar and hoop reinforcement were measured. The demolished columns were divided into 3 segments along their height. For each segment, the corrosion of all the individual bars and hoop reinforcement are measured. The detailed mass loss calculations data are available in a form of Excel spreadsheet data, which is attached to this paper. Table 5 shows the average measured corrosion of each segment, where the length of each segment is measured from the base. The photos of the corroded reinforcement taken out of the corroded column A1 and B1 are shown in Figures 7 and 8. The bar labels shown in the photos are related to the detailed corrosion calculations provided in the attached data file.

Transfer Function Estimate of the Corroded Columns

One of the most popular methods to describe the frequency content of a time-series is Power Spectral Density (PSD) (Chan and Cryer, 2008). The PSD estimates can be used in system identification for structural health monitoring. The periodic pattern (if there is any) of a time-series can be quantified by PSD by calculating the peaks, in frequency, which corresponds to these periodicities. If the excitation and response of a linear system is known, a system identification can be performed by estimating the transfer function (Chan and Cryer, 2008). This system identification method can be used in corroded columns before and after corrosion to identify the impact of corrosion on effective stiffness and dynamic properties of column specimens.

In this study the impact hammer tests (Liu et. al., 2022) is used to estimate the transfer function. Each column was instrumented with two accelerometers in longitudinal and transverse directions. Five impact tests in each direction was performed and the average transfer function is estimated (Verboven, 2005) for each direction. Figure 8 shows the transfer function estimates of column A1 and B1 before and after corrosion.

The frequency associated with the first peak in transfer function is the frequency of the first mode of vibration; i.e. natural frequency of the system. Figure 9 shows that corrosion resulted in an increase in the natural frequency of both columns. Figure 9a shows that natural frequency of the Column A1 is 8.5Hz before corrosion and 9.5Hz after corrosion. Figure 9b shows that natural frequency of Column B1 is 12Hz before corrosion and 14.5Hz after corrosion. This shows that although corrosion results in damage in concrete, the internal volumetric pressure due to the expansion of rust products could increase the stiffness of the column. This observation is important when estimating the initial effective stiffness of columns for seismic assessment and evaluation of corroded columns. The impact of corrosion on the stiffness degradation during the cyclic loading experiments is discussed under the relevant section in this paper.

Experimental Test Results and Discussion

Nonlinear cyclic response of well-confined uncorroded Column A

Figure 10 shows the nonlinear cyclic response of the uncorroded Column A with the failure points corresponding to the first and second fractures of the vertical reinforcement bars marked. The key damage states are illustrated in Figure 11. The flexural cracks due to reinforcement yielding started appearing at about 1.5% drift. At about 2% drift the column base to foundation connection started splitting, which is the sign of reinforcement slip and strain penetration at column base (Figure 11a). As the loading amplitude increased, at about 3% drift, the concrete cover started to crush at the front face of the column (Figure 11b). At 4% drift, significant visible buckling of the vertical reinforcing bars at the front face of the column was observed (Figure 11c). Since the concrete cover crushing started at 3% from the face, this confirms that bar buckling started at lower drift, which resulted in concrete cover spalling. Following the severe bar buckling, the first buckled bar fractured in the next cycle at 4.5% drift (Figure 11d). Finally, in the final cyclic amplitude targeted at 5.5% drift, the second buckled bar fractured at 4.5% drift during the reloading from compression to tension (Figure 11e). The failure mechanism of the buckled bars confirms that both bars fractured during the unloading phase while they were still in compression. This is due to the combination of significant inelastic buckling and low-cycle fatigue of vertical bars, which is in good agreement with findings reported by other researchers (Meda et. al., 2014; Ge et. al., 2020). In EC8 (CEN, 2005) hoop spacing, $S_L \leq 6$ times the longitudinal bar diameter, d_b is suggested. In this experiment, the S_L/d_b ratio was 5, but buckling of the vertical bars and yielding of hoop reinforcement were observed. The experimental results show that the interaction between the stiffness of the hoop reinforcement and the flexural rigidity of the vertical bars is an important factor in seismic detailing of RC columns, which supports the findings reported by other researchers (Dhakal and Maekawa, 2002). This phenomenon is not explicitly captured in the current code, which is an area for further research.

Nonlinear cyclic response of well-confined corroded Column A1

Figure 12 shows the nonlinear cyclic response of the corroded Column A1. The identified failure points are marked on the Figure 12 and the corresponding damages are shown Figure 13. The corrosion was localised at the bottom of the column, and hence, vertical bar slippage and delamination of the column foundation interface occurred at about 2% drift (Figure 13a) similar to Column A (Figure 11a). However, in |Column A1, Most of the column deformation was concentrated at the base of the column, and therefore, not much flexural cracks were observed during the cyclic tests. As drift ratio increased, concrete cover spalled at about 3% drift (Figure 13b), followed by fracture of the first and the second vertical bars at 3.5% drift (Figure 13c). Finally, the third vertical bar fractured at about 4% drift (Figure 13d), which resulted in a complete failure of the column. This failure mode is completely different from the failure mode of the same column without corrosion. This is due to the localised corrosion of a few vertical reinforcement bars at the base of the column. Table 3 indicates that the average corrosion of vertical bars within the 200mm above the foundation is 10.40%. However, the corrosion in bars A1, A2, and A3 (these references are defined in the attached Excel file with detailed mass loss calculations) was 17.77%, 19.11%, 12.03%, respectively, which was localised at the base of the column. This resulted in a premature fracture of these bars at the base of the column.

Nonlinear cyclic response of lightly-confined corroded Column B1

Figure 14 shows the nonlinear cyclic response of the corroded Column B1. The failure points are identified on Figure 14 and the corresponding damages are shown Figure 15. The first visible flexural cracks started to appear at about 0.5% drift with a vertical crack along a corroded bar. The vertical crack was due to the corrosion crack which already existed in the column and its width increased during the test. At about 0.8% drift, premature spalling of the concrete cover on the back face of the column was observed (Figure 15a). This was at the location at about 400mm above the foundation, where concrete cover was partially spalled due to corrosion prior to the cyclic test. At about 2% drift, the first vertical bar fractured due to severe pitting corrosion followed by concrete cover spalling (Figure 15b) during the load reversal from tension to compression. Visible bar buckling was observed at 3% drift (Figure 15c), which was followed by core concrete crushing in the following cycle at 3.5% drift. At 4% drift a corroded hoop fractured, which resulted in core concrete crushing. The significant localised/pitting corrosion at 400mm above the foundation was the point where the first vertical bar fractured. At this location, corrosion resulted in a complete loss of the hoop reinforcement, which resulted in premature concrete cover spalling.

Digital Image Correlation (DIC)

DIC can be employed for measuring crack damage and strain field on the surface of reinforced concrete columns, with a limited number of studies having utilised the method on curved surfaces at present (Al-Kamaki, 2021; Sun et al., 2023). The processed DIC images, showing von-mises equivalent strain contour plots at cracking moment, 1% drift and drift at ultimate strength of Columns A, A1 and B1 are presented in Figure 16. For Column A , the DIC data show that flexural cracks occurred after the first lateral drift cycles (0.1% drift) (Figure 16(a)), beyond which the number and strain of flexural cracks increased at 1% drift (Figure 16(b)) and at drift at ultimate strength (Figure 16(c)). The corroded column A1 demonstrated similar crack development pattern as column A. Furthermore, Column A1 presented, in addition to flexural cracks, a vertical crack which propagated downwards as the lateral drift increased (Figures 16(d), (e) and (f)). Column B1 demonstrated a lower number of flexural cracks during testing (Figures 16(g), (h) and (i)), due to the reduced confinement of this column and similarly to column A1, a singular vertical crack. These vertical cracks are located above the corroded longitudinal reinforcement and occur from the corrosion process. It is shown from the DIC strain plots (Figures 16(a), (d) and (g)) that cracking of the concrete cover occurs at much lower drift values than the first visibly observed cracks.

The DIC images are utilised to obtain strain in the vertical plain of the plastic hinge region from both loading sides of the columns. The vertical strains, from the extreme fibres of the columns in both loading directions, are then computed together, assuming Euler-Bernoulli beam theory, to obtain the position of the neutral axis, the strains on the longitudinal reinforcement and the curvature of the columns. The curvature k_z (1/mm) was evaluated using Eq. (1) (Kashani et. al., 2017), where, ε_2 and ε_1 are the vertical strains in the extreme tensile and compressive fibres of the column, respectively and d is the depth of the column (mm).

$$k_z = \frac{(\varepsilon_2 - \varepsilon_1)}{d} \qquad \textit{\textbf{Eq. (1)}}$$

The moment–curvature relationships up to 1.33% drift (drift at ultimate strength of column B1) for columns A, A1 and B1 are shown and compared in Figure 17. Column A demonstrates a lower cracking moment than the corroded columns, in negative loading directions. This is partially due to the greater initial stiffness of the corroded columns and partially due to construction tolerance. During the construction of the column A, some of the vertical bars have slightly displaced, and hence, vertical bars were not equally spaced around the perimeter. This has resulted in changing the cracking moment of the Column A and different moment-curvature behaviour in positive-negative direction. The corroded columns demonstrated similar moment–curvature behaviour, in the positive loading direction, up to 0.7×10^{-5} 1/mm, beyond which the moment of the column B1 nearly plateaus. Furthermore, the moment of column A surpassed that of the corroded columns at 0.7×10^{-5} 1/mm curvature in the positive loading direction. All columns reached similar curvature in the positive loading direction. In the negative loading direction, column A reaches a greater curvature than the corroded columns at 1% drift and 1.33% drift. However, the moment of column A was less than that of the corroded columns at the same curvature, with column A1 having a greater moment resistance than column B1.

The neutral axis and axial strain on the longitudinal reinforcement was interpolated between the strain values at the extreme fibres of the columns. The neutral axis of both column A1 and B1 initially advanced away from the centre of the column, moving outward in the same direction as the loading direction. In contrast, column A1's neutral axis at small drifts (< 0.3%) had a bias to the positive loading side, beyond these small drifts the neutral axis then started to move outward in the same direction as the loading direction. Columns A, A1 and B1 demonstrated uneven neutral axis values when subjected to positive and negative loading, with a percentage difference in neutral axis values in the positive and negative loading directions at 1% drift of 25.31%, 59.12%

and 104.56%, respectively. However, the percentage difference between neutral axis values in the positive and negative loading directions decreased in columns A, A1 and B1 as drift increased towards the ultimate load to 8.05%, 17.76 and 30.15%, respectively. The interpolated axial strain on the reinforcing bars presented that the first yield of the reinforcing bars occurred at 0.75%, 1% and 0.6% drift for columns A, A1 and B1, respectively. The mean flexural stiffness of the RC column was calculated, for each cyclic loop, from the moment-curvature relationships using Eq. (2) [50].

$$EI_z = \frac{\left|M^{+}_{max,i}\right| + \left|M^{-}_{max,i}\right|}{\left|k^{+}_{z_{max,i}}\right| + \left|k^{-}_{z_{max,i}}\right|} \quad \textit{Eq. (2)}$$

where, EI_z is the flexural stiffness of the column, $M_{max,i}$ is the peak moment in the positive and negative loading direction in kN.mm, and $k_{z\ max,i}$ is the peak curvature in the positive and negative direction for each loop in 1/mm. In order to compare the mean flexural stiffness from the moment-curvature to the secant stiffness, which represents the total stiffness (flexural, shear and slip) from the force-displacement, the flexural and total stiffnesses for each column were normalised to their initial values. It is shown that for column A (Figure 18(a)) that the flexural stiffness degradation is greater than that of the total stiffness degradation, which can be primarily attributed to slipping in the reinforcing bar as shear is negligible in columns of moderate slenderness. However, beyond 5.0×10^{-6} 1/mm curvature the flexural and total stiffness of column A merge closer together. In contrast to column A, column A1 presented that flexural stiffness and total stiffness are nearly identical up to $1.5\text{x}10^{-5}$ 1/mm curvature (Figure 18(b)), indicating that stiffness from reinforcing bar slip is negligible in Column A1. Column B1 (Figure 18(c)) initially demonstrated similar behaviour to column A up to 6×10^{-6} 1/mm curvature, after which flexural stiffness and total stiffness become nearly the same. Furthermore, flexural stiffness degradation of all columns were compared in Figure 18(d), the flexural stiffness values of all columns were normalised to the initial flexural stiffness value of the uncorroded Column A. The initial flexural stiffness was greater and initial stiffness degradation was lesser in the corroded columns than that of the uncorroded column A. However, beyond 4.0×10^{-6} 1/mm curvature, the stiffness degradation of the corroded columns becomes greater than the uncorroded column A, with column B1 and A1 stiffness intersecting column A's stiffness at curvatures of 1.0×10^{-5} and 1.4×10^{-5} 1/mm, respectively.

Impact of corrosion on effective stiffness degradation of RC columns

The effective secant stiffness of the columns for each cyclic loop can be calculated using the Eq. (1).

$$K_{sec} = \frac{|F^{+}_{max,i}| + |F^{-}_{max,i}|}{|\delta^{+}_{max,i}| + |\delta^{-}_{max,i}|} \qquad \textit{Eq. (3)}$$

Where, K_{sec} is the effective secant stiffness of the column in kN/m, $F_{max,i}$ is the peak force in positive and negative direction in kN, and $\delta_{max,i}$ is the peak displacement in positive and negative direction for each loop in m.

In order to compare the stiffness degradation of all the columns, the K_{sec} calculated for each loop in each column is normalise to the initial effective stiffness K_{ses} of the uncorroded column A. The normalised K_{sec} (Figure 19) shows that the initial stiffness of the corroded columns is higher than the uncorroded columns until cycle number 11, which was less than 0.5% drift ratio. However, as drift ratio of the cyclic test increased the stiffness degradation of the corroded columns became more significant than the corroded column. This is due to the more significant concrete damage in corroded specimens under cyclic loading. The stiffness calculations also confirmed that corrosion has resulted in an increase in initial stiffness of the columns, which is in good agreement with the impact hammer test results discussed under the relevant section.

Impact of corrosion energy dissipation capacity

The hysteretic energy dissipation capacity is an important parameter when RC bridges are subject to earthquake loading. Corrosion can have a significant impact on energy dissipation capacity of ageing bridges, and hence, increases the seismic vulnerability of such bridges. The cumulative hysteretic energy dissipation of each column is calculated and normalised to the corresponding total dissipated energy during the cyclic tests as shown in Figure20a-c. The energy dissipation graphs of all columns show that there is almost no energy dissipation until cycle number 11, which is in good agreement with the stiffness degradation results in Figure 19. In order to compare the energy dissipation capacity of corroded and uncorroded columns, Figure 20(d) shows the cumulative energy dissipation of all columns normalised to the total dissipated energy of uncorroded column. Figure 20d shows that the corroded column B1 has the lowest energy dissipation capacity. This is due to the combination of higher average corrosion (compare to column A1), and poor seismic detailing. Although the column A1 experienced fracture of three vertical bars due to localised corrosion at the base, the average corrosion of the same column was lower than corroded column B1, where only one bar fractured in tension. However, corroded column B1 experienced much more severe damage in concrete followed by buckling of vertical bars due to lack of confinement. This can be clearly explained by comparing the nonlinear cyclic response and backbone curves of all three columns in Figure 21. Figure 21 shows that corrosion has a more significant impact on ductility and energy dissipation capacity of RC columns than residual strength.

Impact of corrosion on equivalent viscous damping ratio

The equivalent viscous damping ratio (ξ) represents the combined effects of elastic and hysteretic damping (Blandon and Priestley, 2005). The modelling and calculation of equivalent viscous damping ratio is available in (Zhang et. al., 2017) and is used here. Figure 22 shows the calculated values of ξ for all three columns. Similar to energy dissipation capacity, ξ started increasing after cyclic 11 and gradually decreased after severe damage. This shows that as the hysteretic energy increased the ξ also increases, and after severe cyclic degradation ξ decreases. Figure 22 shows that the ξ in corroded column B1is initially higher than column A and A1, but the maximum of ξ in corroded column B1 is lower than the other two columns. This is due to the corrosion-induced severe damage in concrete, and hence, the damage in concrete results in more initial damping. However, as the drift ratio increases the cyclic degradation results in reduced damping ratio in comparison to columns A and A1.

Conclusion

Three RC bridge piers with different reinforcement details and corrosion were tested under lateral cyclic loading. Column A was a well-confined uncorroded column and column A1 had the same RC detail with corrosion damage. Columns A and A1 were seismically detailed according to EC8. Column B1 was a lightly-confined corroded column representing ageing bridge piers with non-code conforming RC details. The main conclusions of this study can be summarised as follows:

1. The free vibration tests on test specimens showed that the natural frequency of columns increased after corrosion. This might be due to the increased internal pressure at reinforcement and concrete interface, which results in an increase in friction and bond.
2. Following the conclusion 1, the cyclic tests showed that the initial effective stiffness of the corroded columns was more than the uncorroded specimen. However, as soon as drift ratio increased the stiffness degradation of corroded specimens was more significant than the uncorroded specimen.
3. The uncorroded column was seismically detailed according to EC8 criteria. However, significant inelastic buckling followed by low-cycle fatigue fracture of vertical bars was observed. This phenomenon is due to the interaction of hoop reinforcement and vertical bars, which is not explicitly captured in the current seismic design codes. This is an area for further research.
4. Non-uniform corrosion had a significant impact on failure mechanism of corroded specimens. Corrosion in well-confined column A1 was concentrated at the base of the column, and hence, column failure was governed by localised fracture of bars at the base of the column. Corrosion was more evenly distributed in Column B1

with some localised corrosion at about 200mm above the foundation. This has resulted in significant damage in concrete, followed by inelastic buckling and fracture of vertical bars.

5. Corrosion had a more significant impact on ductility and energy dissipation capacity loss than strength loss of corroded columns. The tests results showed that corrosion resulted in about 5% loss of strength in column A1 and 20% loss of strength in column B1. However, it resulted in about 30% reduction in energy dissipation capacity in column A1 and 60% loss of energy dissipation capacity in column B1.
6. The results showed that DIC data can be used to measure strain field and surface concrete damage at small drift ratios. However, it was observed that use of DIC can challenging on curved surfaces. Therefore, multiple cameras are required for reliable data measurement.

Data Availability Statement

All data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

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Table 1 Experimental Test Matrix

Column ID	Design	28 Days Cube Mean Strength	Estimated Mass Loss
Column A	Well-Confined	75.4 MPa	0
Column A1	Well-Confined	73.7 MPa	20%
Column B1	Lightly-Confined	62.6 MPa	20%

Table 2 Mechanical properties of uncorroded steel reinforcement

Reinforcement Type		8mm (B8)	16mm (B16)
Yield Strength	f_y (MPa)	520	530
Modulus of Elasticity	E_s (MPa)	200426	193913
Yield Strain	ε_y	0.00261	0.00273
Ultimate Tensile Strength	f_u (MPa)	645	640
Strain at Ultimate Tensile Strength	ε_u	0.057	0.165
Fracture Strain	ε_f	0.152	0.227
Unit Mass	m (kg/m)	0.396	1.579

Table 3 Concrete mix in 1 meter cube (water/cement ratio = 0.39)

Mix constituent	Quantity
Cement (52R)	420 kg
4-10mm stone (flint)	901 kg
0-4mm sand	823 kg
Superplasticiser	1.8 L
total water	160 kg

Table 4 Cameras, settings and parameters used in digital image correlation

	Stereo DIC	
Column Side	Negative loading direction side	Positive loading direction side
Camera		
Sensor & digitization	CCD 2456×2058 pixels, 8-bit	CCD 2456×2058 pixels, 8-bit
Exposure time & recording rate	19000 µs, 1 Hz	19000 µs, 1 Hz
Mean camera noise (% of dynamic range)	0.0037%; 0.0034%	0.0033%; 0.0034%
Lens & imaging distance	Nikkor 50 mm, 2.69 m	Nikkor 28 mm, 1.55 m
Number of images averaged for resolution calculation	2	2
Pixel size	3.45 µm	3.45 µm
Region of interest & field of view	100×200 mm, 442×370 mm	100×200 mm, 537×450 mm
Processing		
Subset, step	59, 16 pixels	59, 16 pixels
Matching criterion, interpolation, shape function	ZNSSD, Local Bicubic Spline, Quadratic	ZNSSD, Local Bicubic Spline, Quadratic
Pre-smoothing	None	None
Mean displacement resolution	0.0470 mm [1.45 pixels]	0.0586 mm [1.22 pixels]
Strain		
Smoothing technique	None	None
Virtual strain gauge	7 pixels (24.15 µm)	7 pixels (24.15 µm)
Mean strain resolution	471 µε	366 µε

Table 5 Measured mass loss in corroded columns

Test Specimen	Percentage Mass Loss in the reinforcement (%)					
	Longitudinal Bars			Transverse Ties		
	Segment A*	Segment B*	Segment C*	Segment A*	Segment B*	Segment C*
Column A1	10.40%	5.32%	4.04%	21.78%	19.75%	17.18%
Column B1	12.98%	16.31%	12.68%	28.30%	39.63%	26.27%

* Segment A (0 – 200mm), Segment B (200mm – 400mm), and Segment C (400mm – 600mm)

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Figure 15 Observed damage during the cyclic test of column B1: (a) concrete cover crushing/spalling at 0.8% drift, (b) fracture of vertical bar and severe cover spalling at 2% drift, (c) visible buckling at 3% drift, (d) severe core concrete crushing at 3.5% drift, and (e) hoop fracture at 4% drift

Figure 16 DIC strain images at cracking moment, 1% drift and drift at ultimate strength of Columns A ((a), (b) and (c)), A1 ((d), (e) and (f)) and B1 ((g), (h) and (i)).

Figure 17 Moment – curvature response of columns A, A1 and B1 up to 1.33% drift.

Figure 18 Comparison of normalised mean stiffness from moment-curvature (DIC) and force-displacement responses ((a) – (c)), and comparison of stiffness from moment-curvature between uncorroded and corroded columns (d)

Figure 19 Normalised effective stiffness of all columns

Figure 20 Hysteretic energy dissipation: (a) Uncorroded Column A, (b) Corroded Column A1, (c) Corroded Column B1, and (d) All columns

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Figure 22 Equivalent Viscous Damping Ratio (ξ)