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Seismic Performance of Flexural Reinforced Concrete Columns with Corroded Reinforcement

by Ngoc Son Vu and Bing Li

This paper presents an experimental study carried out on eight full-scale uncorroded and corroded reinforced concrete columns that were subjected to simulated seismic loading to investigate the effects of reinforcement corrosion on the seismic behavior of columns that failed in flexure. The main variables are the corrosion levels of reinforcement and applied axial force ratios. The seismic performance of test specimens was assessed in terms of crack pattern, hysteretic response, yield displacement, lateral load resistance, drift capacity, displacement ductility, and energy dissipation capacity. The results highlighted that the corroded columns suffered a significant degradation of strength and lateral drift capacity as compared to uncorroded columns. In addition, it is noteworthy to observe that the failure mode of corroded columns switched from flexural failure to flexural-shear failure and sudden axial failure when they are highly corroded and subjected to high axial force. An empirical equation based on the maximum pit depth of corroded longitudinal reinforcing bars was proposed to predict the residual flexural strength of corroded columns. Furthermore, based on the shear strength model proposed in ASCE/SEI 41-13, methods were proposed to estimate the shear strength of corroded columns failed in flexural-shear failure and axial failure, which consider the contribution reductions to shear strength of transverse reinforcement and concrete due to corrosion. Among these methods, the shear strength of corroded columns estimated based on the corrosion mass loss and average residual cross-sectional area was not conservative as compared to the test results. In contrast, the shear strength of corroded columns predicted using the minimum residual cross-sectional area of transverse reinforcing bars showed good and reliable correlation with the experimental results.

Keywords: corrosion; flexural strength; reinforced concrete column; reinforcement; shear strength.

INTRODUCTION

Reinforcement corrosion has been found to be one of the most significant causes of deterioration for reinforced concrete (RC) structures subjected to various corrosive environmental actions. The corrosion products expand the volume of corroded steel reinforcement, leading to tensile stresses that develop at the interfacial regions between steel reinforcement and concrete, causing the cover concrete of these RC structures to crack and spall off. In addition, the deterioration of strength and deformation capacity of reinforcement caused by corrosion also results in significant reduction of confinement effectiveness for confined concrete and buckling resistance of longitudinal reinforcement. As a result, the performance of RC structures will be adversely affected, even more so for structures under severe corrosive environments and subjected to seismic loading. However, in the literature, the experimental research about these influ-

ences of reinforcement corrosion on the seismic response of RC columns is minimal. Among the limited research, studies^{1,2} investigated the effects of corroded longitudinal reinforcement on the seismic behavior of flexural-dominated columns. The test results in these studies showed that, although the flexural strength and deformation capacity significantly deteriorated when the corrosion level of reinforcement increased, the failure mode did not change as compared to uncorroded columns. It was noted that in these studies, only longitudinal reinforcing bars were corroded, which did not simulate the corrosion phenomenon of RC columns in reality, where the transverse reinforcement is even more corroded because of its smaller diameter. In addition, its distance to the corrosive environment is also smaller. Other studies³⁻⁵ focused on the effectiveness of strengthening methods on corroded columns under cyclic loading. Therefore, there is a shortfall of research on the seismic performance of RC columns with both corroded transverse and longitudinal reinforcing bars, particularly in terms of investigations on their strength and deformation capacity deterioration, especially the possibility of change in failure modes due to severe corrosion and high applied axial forces. This paper presents an experimental investigation carried out on eight full-scale uncorroded and corroded columns under cyclic loading. To accelerate the corrosion process in test specimens, the electrochemical method is used by applying an externally impressed current on both longitudinal and transverse reinforcing bars. After that, eight full-scale uncorroded and corroded specimens with various corrosion levels are tested under constant axial force and quasi-static horizontal cyclic load. The experimental results highlighted that the corroded columns displayed a significant degradation of strength and deformation capacity compared to uncorroded columns, especially as the failure mode switched from typical flexural failure to flexural-shear failure, and sudden axial failure when they were highly corroded and subjected to high axial load ratio.

RESEARCH SIGNIFICANCE

This study contributes to the comprehensive understanding of the seismic response of RC columns in corrosive environments, especially the possibility of change in their

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Fig. 1—Test specimens: (a) reinforcement details; and (b) deformation measurement. (Note: All dimensions are in mm; 1 mm = 0.0394 in.)

failure modes due to severe corrosion and high applied axial forces. An empirical equation based on the maximum pit depth of corroded longitudinal reinforcing bars is proposed to predict the residual flexural strength of corroded columns. Furthermore, an assessment method is also developed to predict the shear strength of corroded columns, taking into consideration the contribution reductions of transverse reinforcement and concrete to the shear strength deterioration by incorporating the corrosion effects into the shear strength model proposed in ASCE/SEI 41-13.⁶

EXPERIMENTAL PROGRAM

Specimen details

This experimental study included eight full-scale uncorroded and corroded RC columns tested with combined axial force and cyclic loading. Two of the columns acted as control specimens without corrosion of reinforcement, tested with constant axial loads of $0.1f'_cA_g$ and $0.25f'_cA_g$ together with horizontal cyclic loading. The other six columns were subjected to accelerated corrosion for different time periods, resulting in various corrosion levels. After the accelerated corrosion process, three corroded columns were tested with axial load of $0.1f'_cA_g$ while the axial load of $0.25f'_cA_g$ was applied for the other three corroded columns. Figure 1(a) shows the specimen's concrete, arrangements of longitudinal reinforcing bars, and transverse hoops designed for seismic zone requirements in accordance with ACI 318-14,⁷ which is sufficient confining transverse reinforcement in the plastic hinge regions. All test specimens were designed to have a similar cross section of 350 x 350 mm (13.8 x 13.8 in.) and clear height of 1780 mm (70.1 in.), resulting in the same shear span-to-depth ratio a/d of 3.18. In this study, the shear span a is equal to half of the clear height of the column due to double curvature test, while the effective depth d can be assumed to be $d = 0.8h$, where h is the depth of the column cross section.⁶ The target concrete strength of 35 MPa (5075 psi) for test specimens was supplied by a

local concrete plant and casted in eight batches. Three 150 x 300 mm (5.9 x 11.8 in.) cylinder specimens were made for each batch. The average compressive strengths of the cylinders at testing days, above 28 days, for the eight batches of specimens are indicated in Table 1. The transverse reinforcement of R8, with a yield strength f_{yt} of 300 MPa (41.5 ksi) and measured diameter of 7.8 mm (0.3 in.), was used for all test specimens with 135-degree bent, spaced at 50 mm (1.97 in.) for the plastic hinge regions and 100 mm (3.9 in.) for the middle regions, while eight deformed reinforcing bars T20 with a yield strength f_{yt} of 550 MPa (80 ksi) and measured diameter of 20 mm (0.8 in.) were used for longitudinal reinforcing bars. Table 2 summarizes the properties of reinforcement for test specimens.

Accelerated corrosion scheme

The electrochemical method was adopted to accelerate the corrosion process in RC specimens, as the natural corrosion process usually takes several years or decades. Figure 2 schematically demonstrates the accelerated corrosion of both longitudinal and transverse reinforcing bars using the electrochemical method. A constant current density of approximately 0.5 mA/cm² was applied to the steel reinforcing bars that were connected to the anodes of 12 DC power supplies while the copper plates were used as the cathodes. Inside the water tank, 12 copper plates were then placed equally along the length of the test specimens at both their left side and right side. To simulate a corrosive environment, the specimens were immersed in the water tank with a 5% sodium chloride (NaCl) solution, as shown in Fig. 2. In theory, the corrosion level in terms of mass loss can be estimated using Faraday's law and governed by the accelerated corrosion time. However, many researchers pointed out that Faraday's law overestimates the corrosion level, and this finding is also confirmed herein. Therefore, in this study, the corrosion level was estimated based on the measured data of the experiment, as presented in the following section.

Fig. 2—Accelerated corrosion scheme.

Table 1—Summary of test specimens and corrosion measurement

Specimen	f'_c , MPa	$P/f'_c A_g$	Accelerated corrosion time, days	Transverse reinforcement				Longitudinal reinforcement				Cover concrete	
				$X_{corr}^{(1)}$, %	$X_{corr}^{(2)}$, %	$X_{corr}^{(3)}$, %	$Pit_{max}^{(t)}$, mm	$X_{corr}^{(1)}$, %	$X_{corr}^{(2)}$, %	$X_{corr}^{(3)}$, %	$Pit_{max}^{(l)}$, mm	Maximum crack width, mm	Total crack width, mm
U1	32.2	0.1	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
C1	37.5	0.1	30	11.5	23.8	48.7	2.58	3.4	7.1	8.0	0.82	0.2	0.26
C2	32.0	0.1	60	14.4	21.3	51.9	2.62	6.1	10.4	13.1	1.36	0.2	0.18
C3	40.0	0.1	90	25.0	30.5	62.3	3.06	10.9	20.4	27.6	3.05	1.4	1.19
U2	38.1	0.25	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
C4	28.8	0.25	30	13.6	22.6	41.3	1.91	3.4	4.2	6.1	0.62	0.3	0.52
C5	31.3	0.25	60	15.5	29.5	59.2	3.06	3.9	8.0	10.0	1.03	0.5	0.48
C6	37.5	0.25	90	23.5	33.5	73.8	3.79	7.3	11.5	14.3	1.49	1.0	0.98

Notes: f'_c is compressive concrete strength; P is applied axial force; A_g is gross cross-sectional area of column; $Pit_{max}^{(t)}$ and $Pit_{max}^{(l)}$ are maximum corrosion pit depths of transverse and longitudinal reinforcing bars, respectively; 1 MPa = 0.145 ksi; 1 mm = 0.04 in.

Corrosion level assessment

It is well known that the corrosion degree of reinforcement can be evaluated in terms of either mass loss or residual cross-sectional area. In this study, three evaluative methods of corrosion degree of reinforcement based on experimental measurement are proposed, including corrosion levels in terms of mass loss $X_{corr}^{(1)}$, average residual cross sectional area $X_{corr}^{(2)}$, and minimum residual cross sectional area, or pitting corrosion $X_{corr}^{(3)}$, defined as follows

$$X_{corr}^{(1)} = \frac{W_0 - W_1}{W_0} \quad (1)$$

$$X_{corr}^{(2)} = \frac{D_0^2 - D_{re}^2}{D_0^2} \quad (2)$$

$$X_{corr}^{(3)} = \frac{D_0^2 - D_{min}^2}{D_0^2} \quad (3)$$

where W_0 and D_0 are the original weight and diameter of reinforcement before corrosion, respectively; and W_1 , D_{re} , and D_{min} are the residual weight, average residual diameter, and minimum residual diameter of reinforcement after

Table 2—Properties of reinforcement

Steel type	Measured diameter, mm	Yield strength, MPa	Ultimate strength, MPa	Modulus of elasticity, GPa
R8	7.8	300	465	210
T20	20	550	635	210

Notes: 1 mm = 0.04 in.; 1 MPa = 0.145 ksi.

corrosion, respectively. The measurement procedure follows ASTM G1-03,⁸ particularly after testing, that all the reinforcing bars were taken out of the test specimens and then were cleaned with steel brushes. After that, they were immersed in 10% hydrochloric acid (HCl) for 20 minutes to remove all the rust. Finally, all the reinforcing bars were measured to estimate their weight loss and residual diameters. To measure the average and minimum residual diameters (or pitting corrosion) along the length of transverse reinforcing bars, the transverse reinforcement was equally divided into eight regions—four regions at the corners, and the other four regions at the middle of steel reinforcement. A digital vernier caliper was used to measure the average and minimum residual diameters of the corroded reinforcement. Eight regions of each transverse reinforcing bar were measured to obtain their residual diameters. After that, the average and

Fig. 3—Appearance of some typical corroded transverse and longitudinal reinforcing bars of corroded column C1 after testing and removing all rust.

Fig. 4—Appearance of some typical corroded transverse and longitudinal reinforcing bars of corroded column C3 after testing and removing all rust. (Note: East-west is lateral loading direction. Dimensions in mm; 1 mm = 0.0394 in.)

minimum residual diameters of each transverse reinforcing bar were obtained by taking the average and minimum values of these obtained values of eight regions, respectively. The same measurement procedure was repeated for all transverse reinforcing bars. Finally, the average and minimum residual diameters of transverse reinforcement of the specimen were achieved by taking the average value of average and minimum residual diameters of all transverse reinforcing bars, respectively. The average and minimum residual diameters of longitudinal reinforcing bars were also measured in a similar manner by equally dividing each longitudinal reinforcing bar into 12 regions of approximately 150 mm (5.9 in.). As observed, the corrosion level of outer transverse reinforcement is much higher than that of inner transverse reinforcement and longitudinal reinforcement because its distance to the outside environment is smaller and its diameter is also smaller than that of longitudinal reinforcement. For example, the average corrosion level in terms of mass loss of outer transverse reinforcement is approximately 2.2, and 4.5 times higher than that of inner transverse reinforcement and longitudinal reinforcement, respectively. The measured corrosion levels of transverse and longitudinal reinforcing bars of corroded specimens are tabulated in Table 1. The experimental observation also exposed that the pitting corrosion of both transverse reinforcement and longitudinal reinforcement becomes more serious with the increase of severity of corrosion. For the corroded columns subjected to low corrosion levels, the diameter of reinforcement reduced relatively uniformly, generally defined as uniform corrosion, as demonstrated in Fig. 3 for corroded Specimen C1 with corrosion levels in terms of mass loss of 11.5% and 3.4% for

stirrups and longitudinal reinforcing bars, respectively. On the other hand, with the highly corroded columns, the pitting corrosion of both transverse and longitudinal reinforcing bars was noticeable; even the fracture of transverse reinforcing bars induced by corrosion can be observed, as indicated in Fig. 4 for corroded Specimen C3 with corrosion levels in terms of mass loss of 25.0% and 10.9% for stirrups and longitudinal reinforcing bars, respectively.

Corrosion cracks

After the accelerated corrosion, all corroded specimens showed visible cracks. Their width and length were traced and measured at four sides using crack width gauges. Figure 5 demonstrates the distribution of cracks induced by corrosion on the four sides of corroded columns. It is also observed that the cracks induced by corrosion primarily appeared and developed vertically along the direction of longitudinal reinforcement. To consider the effect of cover concrete cracks to the seismic behavior of corroded RC columns, the total crack width of the specimen W_{cr} , which is defined as the average value of crack width determined for the four sides of the test specimen, is determined as follows^{9,10}

$$W_{cr} = \frac{\sum w_{cri} l_i}{L} \quad (4)$$

where w_{cri} is the width of crack i ; l_i is the length of the crack i ; and L is the clear height of test specimens ($L = 1780$ mm [70.1 in.]). The total crack width and maximum crack width of the specimens are indicated in Table 1, which can be easily measured in the experiment and in actual practice using the crack width gauges.

Fig. 5—Corrosion-induced concrete cover crack.

Test setup

Figure 6 demonstrates the test setup for cyclic loading that enables the test specimens to be performed under double-curvature bending. The axial force is applied constantly using two vertical actuators with a capacity of 1000 kN (225 kip) through the L-shaped steel frame. A horizontal actuator is used to produce the reversible horizontal displacement and its axis passes through the midheight of the specimen to simulate the double-curvature bending condition. The quasi-static cyclic loading history for test specimens is shown in Fig. 7. During the test, several cycles of lateral displacements corresponding to drift ratio from 0.25 to 6.0% were applied through the horizontal actuator. The drift ratio (DR) is defined as the ratio of interstory horizontal displacement over the story height. In this study, the column specimens are subjected to two cycles of loading with displacement equal to story drift ratio of 0.25%, 0.5%, 1%, 1.5%, 2%, 2.5%, 3%, 3.5%, 4%, 4.5%, 5%, 5.5%, and 6%, as shown in Fig. 7. The horizontal displacement is measured using linear variable displacement transducers (LVDTs) located at the top of specimens and parallel to the horizontal actuator. Numerous LVDT sets are also installed along the height of specimens to measure the slip, shear, and flexural deformation, as demonstrated in Fig. 1(b). In this study, no strain gauges are installed on the reinforcing bars because they are damaged by the accelerated corrosion process.

EXPERIMENTAL RESULTS AND DISCUSSION

This section presents and discusses the experimental results of eight uncorroded and corroded specimens tested with various corrosion levels and axial load ratios. Figure 8

Fig. 6—Experimental setup.

Fig. 7—Quasi-static cyclic loading history for test specimens.

Fig. 8—Hysteric response of test specimens. (Note: V_u is theoretical flexural strength; V_n is nominal shear strength; PL1 is yielding level; PL2 is maximum lateral force; PL3 is failure stage corresponding to maximum lateral force dropped by 20%; and 1 mm = 0.04 in.)

demonstrates the hysteretic responses of all test specimens. Three performance levels were used to highlight the response of test specimens, particularly in terms of yielding level (PL1), maximum lateral force (PL2), and failure stage (PL3, which is defined corresponding to the maximum lateral force dropped by 20%). In general, the hysteretic responses showed the deterioration of lateral load resistance and the pinching effect during repeated cyclic loading that was more obvious with corroded columns due to the bonding reduction between reinforcement and surrounding concrete induced by corrosion. As expected in design, uncorroded U1 and U2 failed in typical flexure; these specimens did not reach the nominal

shear strength estimated following the suggestion of ASCE/SEI 41-13.⁶ However, their theoretical flexural strengths were attained, which were estimated using the test results of materials and in accordance with ACI 318-14,⁷ as indicated in Fig. 8 for uncorroded U1 and U2. Seismic behavior assessment of all test specimens is presented in the following section.

Crack pattern and failure modes

The appearance and propagation of cracks on the surface of test specimens were recorded at every maximum applied horizontal displacement during cyclic loading. Figures 9 and 10 demonstrate the crack patterns of test specimens at

Fig. 9—Crack patterns of test specimens at maximum lateral force.

Fig. 10—Crack patterns of test specimens at final stage (end of test).

maximum lateral force (PL2) and final stage (end of the test), respectively. As observed, three failure modes were identified, in which the first failure mode is typical flexural failure, which happened in uncorroded U1 and U2 and corroded C1 and C2 subjected to low corrosion levels and an axial force of $0.1f_c'A_g$. For these specimens, the horizontal flexural cracks perpendicular to column axis appeared first in the top and bottom regions of the specimens at drift ratios of approximately 1.0% and 0.75% for uncorroded specimens and corroded specimens, respectively. When the applied drift ratio increased, these flexural cracks extended within the plastic hinge regions of these specimens and the appear-

ance of flexural-shear diagonal cracks was also observed. However, these flexural-shear cracks did not lead to the failure of these specimens. It was noted that, compared to the uncorroded U1, the corroded C1 and C2 showed less flexural and flexural-shear cracks. In addition, for these corroded specimens, together with the flexural cracks, the vertical cracks along the longitudinal reinforcement were also obvious. This is because the initial cracks induced by corrosion developed vertically along the direction of longitudinal reinforcement, and afterward, these cracks further developed and widened with the increasing applied horizontal displacement and axial force. As commonly observed in the domi-

nated flexural columns, the test specimens failed with extensive spalling of cover concrete and crushing of core concrete within the plastic hinge regions of the specimens. It is also noted that none of the stirrups fractured at the end of the test for corroded C1 and C2. The second failure mode defined as flexural-shear failure was observed in corroded C3, C4, and C5, which were subjected to either high corrosion levels or high axial load ratio. For these specimens, fewer flexural cracks were observed at the early stages of cyclic loading compared to those that failed in typical flexure. On the other hand, the diagonal shear cracks were more obvious for these corroded specimens, and right after the maximum force was attained, extensive diagonal cracks developed in the plastic hinge regions of the specimens with the angles of main diagonal cracks of approximately 40, 45, and 30 degrees for corroded C3, C4, and C5, respectively. These diagonal cracks directly resulted in the failure of these specimens. It is also noted that the development of vertical cracks along the longitudinal reinforcement was more obvious than that of corroded C1 and C2 because these corroded specimens were subjected to higher corrosion levels. This is evident of the bonding reduction between reinforcement and surrounding concrete induced by corrosion, which mainly caused the adverse pinching effect during repeated cyclic loading. This phenomenon was also more noticeable for corroded C4 and C5 subjected to a high axial force of $0.25f_c'A_g$, as indicated in Fig. 8. The fracture of stirrups at the end of the test was also prominent in these corroded specimens, which significantly caused the change of failure mode from typical flexural failure to flexural-shear failure. The third failure mode was identified as sudden axial failure with the explosion of concrete and reinforcement, which happened at the top end of corroded C6 under the high corrosion level of transverse reinforcement of 23.5% and high axial force of $0.25f_c'A_g$, as shown in Fig. 10. It was also observed that after taking all reinforcement out of the test specimens, the majority of transverse reinforcement fractured due to extremely severe corrosion, resulting in the loss of confinement effectiveness for confined concrete and buckling resistance of longitudinal reinforcement, which mainly caused the sudden axial failure.

Hysteretic response comparisons

Figure 11 shows the comparisons of hysteric responses between uncorroded specimens and corroded specimens that are subjected to the same axial forces of $0.1f_c'A_g$ and $0.25f_c'A_g$. As observed, the uncorroded U1 and U2 failed in typical flexure, which performed in a ductile manner, while the corroded C3, C4, and C5 failed in flexural-shear and responded in a more brittle manner—that is, the lateral load resistance deteriorated right after the maximum lateral force was attained and the pinching effect was obvious during repeated cyclic loading. Comparisons of the backbone curves of test specimens with various corrosion levels and axial load ratios are also presented in Fig. 12. Of note, from henceforth, the values of yield displacement, lateral load capacity, displacement ductility, drift ratio, and energy dissipation are the average values of positive and negative loading directions, as indicated in Table 3.

Yield displacement—In this paper, the yield displacement of all test specimens is defined theoretically as follows¹¹: a secant is drawn at the point of 70% of the maximum lateral force on the lateral force-horizontal displacement curve, and another horizontal line corresponding to the maximum lateral force is also drawn. From the intersection of these two lines, taking the projection to the horizontal axis, the yield displacement is obtained, as demonstrated in Fig. 8 (Specimen U1). Table 3 indicates the average values of yield displacement for test specimens from positive and negative loading directions. As indicated, the corrosion level of reinforcement and applied axial force significantly affected the yield displacement of corroded columns compared to uncorroded columns. For instance, compared to uncorroded U2 and subjected to the same axial load of $0.25f_c'A_g$, the yield displacement reduced 37.1%, 48.0%, and 24.4% for the corroded C4, C5, and C6, respectively. The similar reduction trend can be also observed for those corroded specimens under axial load of $0.1f_c'A_g$.

Lateral load capacity—Table 3 indicates the values of the lateral load capacity of test specimens, which were normalized as $V_{max}/A_g\sqrt{f_c'}$ due to the varied compressive concrete strength among test specimens. It was observed that the lateral load capacity decreased corresponding to the increase of corrosion levels. For example, in the test specimens with an axial load of $0.25f_c'A_g$, the normalized lateral load capacity of these specimens reduced 5.0%, 12.5%, and 17.5% from uncorroded U2 to the corroded C4, C5, and C6, respectively. Similar deterioration of normalized lateral load capacity for U1, C1, C2, and C3 with axial load of $0.1f_c'A_g$ can be also observed with reductions of 3.0%, 15.2%, and 24.2%, respectively.

Displacement ductility—Generally, displacement ductility is an important parameter to estimate the shear strength of RC columns, which is the ratio of ultimate displacement to yield displacement (defined previously). As commonly used in literature, ultimate displacement is the measured displacement when the applied lateral force reduces to 80% of the maximum lateral force. Table 3 indicates the values of displacement ductility of test specimens. As predicted, the displacement ductility reduced with the increase of corrosion level, even more so for corroded specimens under high applied axial load. For example, with the test specimens under axial force of $0.25f_c'A_g$, reductions of displacement ductility of 16.7%, 14.3%, and 45.2% can be observed for corroded C4, C5, and C6 as compared to uncorroded U2, respectively.

Drift capacity—The values of drift capacities of test specimens at various performance levels are presented in Table 3, including drift ratios at the yielding level, maximum lateral force, and failure stage. Overall, drift capacities decreased when the corrosion level and axial load ratio increased at all performance levels of test specimens. Regarding the drift ratio at failure stage (PL3), a reduction of 11.5%, 19.6%, and 33.4% was observed for corroded C1, C2, and C3 compared to the uncorroded U1, respectively. More significant reduction of the drift ratio at failure stage can be observed for highly corroded C4, C5, and C6, which were subjected to a high axial force of $0.25f_c'A_g$ —the reductions were 47.4%, 55.4%, and 57.9% compared to uncorroded U2, respec-

Fig. 11—Comparisons of hysteretic responses of uncorroded specimens and corroded specimens. (Note: 1 mm = 0.04 in.)

Fig. 12—Comparisons of backbone curves of test specimens subjected to: (a) axial force of $0.1f_c'A_g$; and (b) axial force of $0.25f_c'A_g$. (Note: 1 mm = 0.04 in.)

tively. This is due to the fracture of transverse reinforcement

induced by severe corrosion resulting in the significant deterioration of the drift capacity at failure stage.

Table 3—Comparisons between uncorroded columns and corroded columns with various corrosion levels and axial load ratios

Specimen	f'_c , MPa	$P/f'_c A_g$	Yield displacement, mm	Yield curvature, $1/\text{mm} \times 10^{-6}$	V_{max} , kN	$\frac{V_{max}}{A_g \sqrt{f'_c}}$	δ_y , %	δ_m , %	δ_s , %	μ_δ	Energy dissipation (kN·m) at			Failure modes
											PL1	PL2	PL3	
U1	32.2	0.1	25.0	9.02	232.1	0.33	1.4	3.3	5.8	4.1	3.6	15.5	40.9	1
C1	37.5	0.1	22.7	5.61	238.8	0.32	1.3	2.5	5.1	4.0	3.5	11.0	32.5	1
C2	32.0	0.1	24.8	4.92	193.2	0.28	1.4	2.5	4.7	3.3	2.7	7.0	20.6	1
C3	40.0	0.1	18.9	7.28	190.7	0.25	1.1	1.8	3.9	3.6	2.1	4.1	11.8	2
U2	38.1	0.25	25.4	8.42	302.5	0.40	1.4	2.7	6.0	4.2	4.5	14.2	51.9	1
C4	30.0	0.25	16.0	4.07	254.4	0.38	0.9	2.4	3.2	3.5	3.5	12.0	19.7	2
C5	31.3	0.25	13.3	5.14	237.1	0.35	0.7	2.0	2.7	3.6	1.3	7.9	12.4	2
C6	37.5	0.25	19.2	7.26	250.0	0.33	1.1	2.0	2.5	2.3	3.8	7.4	11.5	3

Notes: δ_y , δ_m , and δ_s are drift ratios at yielding level, maximum lateral force, and failure stage, respectively; and μ_δ is displacement ductility. Failure modes 1, 2, and 3 stand for flexural failure, flexural-shear failure, and axial failure, respectively. 1 mm = 0.04 in; 1 MPa = 0.145 ksi; 1 kN = 0.225 kip.

Energy dissipation capacity—Energy dissipation capacity of RC columns subjected to cyclic loading is defined as the area under the lateral force and horizontal displacement hysteretic curves. Table 3 presents the values of energy dissipation capacity of test specimens at different performance levels. As observed, the total energy dissipation capacity depends on not only corrosion level of test specimens but also the applied axial force. For example, with the increase of corrosion level, the total energy dissipation at failure stage decreased 20.6%, 49.7%, and 71.1% from uncorroded U1 to corroded C1, C2, and C3, respectively. A similar reduction trend of energy dissipation capacity was also observed for corroded specimens subjected to an axial force of $0.25f'_c A_g$. In addition, it was observed that the energy dissipation capacity among corroded specimens with similar corrosion levels of reinforcement was also reduced with the increase of applied axial force from $0.1f'_c A_g$ to $0.25f'_c A_g$.

Curvature distribution

In this study, the experimental curvatures of test specimens are estimated based on the test data of LVDT sets as follows¹²

$$\phi = \frac{\Delta_t - \Delta_c}{h_t l_t} \quad (5)$$

where Δ_t and Δ_c are the displacement of the LVDTs measured by the tension and compression sides of the column, respectively; l_t is the length of column segment where one set of two LVDTs is attached; and h_t is the center-to-center distance between two LVDTs. Figure 13 demonstrates the curvature distributions of test specimens at yield displacement (as defined in previous section) and maximum lateral force for both positive and negative loading directions. As observed, for both uncorroded and corroded columns, the curvatures at yield displacement and maximum lateral force distributed relatively linear along 5.6% to 94.4% column height. In addition, the main contribution to the flexural deformation of all test specimens was observed from their top and bottom ends. Table 3 indicated the curvatures of test specimens at yield displacement (defined as yield curvature). As indicated, the

increase of corrosion level resulted in the decrease of yield curvature for the test specimens subjected to both axial forces of $0.1f'_c A_g$ and $0.25f'_c A_g$. It is noted that these measured yield curvatures due to flexural deformation at the top and bottom ends of the columns were obtained by deducting the curvature due to bar slip from the total measured curvatures.

Displacement composition

Contributions of the slip, shear, and flexural deformation to the total lateral displacement at the top of test specimens were estimated based on the reading results of the attached LVDTs. The slip and flexural deformation was estimated using the average curvatures, which were indicated in the previous section, while the change in length of the pairs of diagonal LVDTs attached to the side of the specimens was used to calculate the shear deformation. Figure 14 shows the displacement composition for test specimens in which the contributions of the slip, shear, and flexural deformation are presented from the upper part to the lower part, respectively. The experimental results showed that the contribution of flexural deformation to the total displacement was the most dominant for test specimens because they failed in flexure or flexural-shear. In addition, the slip deformation was also a relatively large proportion of the total displacement for all test specimens. As observed, with the increase of the applied horizontal displacement, the contribution of flexural deformation to total displacement reduced while the contribution of slip deformation increased, which was more obvious for corroded specimens because of the reduction of bond slip between longitudinal reinforcing bars and concrete at their top and bottom ends. On the other hand, for all test specimens, the contribution of shear deformation was found to be relatively constant corresponding to the increase of horizontal displacement with average values from 10% to 25% of total displacement. In addition, it is worth noting that for the corroded C3, C4, and C5, in which the failure mode switched from flexure to flexural-shear, the increase of contribution of shear deformation can be observed after the maximum lateral force was attained, as shown in Fig. 14. This observation is consistent with the appearance and propagation of diagonal shear cracks within the plastic hinge regions of these corroded specimens.

Fig. 13—Curvature distribution of test specimens.

Fig. 14—Displacement decomposition of test specimens.

Proposed residual flexural strength estimation

A comparison of the theoretical flexural strength of two uncorroded specimens that failed in flexure with the experimental result of the maximum lateral force shows a good correlation with the ratios of the test result to the theoretical flexural strength of uncorroded U1 and U2 estimated based on sectional analysis: 0.97 and 1.04, respectively. Therefore, it is reasonable to set the flexural strength of test specimens equal to their maximum lateral force measured in the experiment. It is also noted that the flexural strength of corroded RC

members mainly reduces because of the reduction of mechanical properties of corroded reinforcement—yield strength and cross-sectional area, bonding between reinforcement and concrete, and corrosion-induced cover concrete crack. To consider the complicated effects of these parameters on the reduction of flexural strength of corroded RC members, several studies^{13,14} in the literature suggested that the residual flexural strength of corroded beams can be predicted using the empirical expressions in which the ratio of the maximum pit depth to the original radius of longitudinal reinforcement

Table 4—Proposed residual flexural strength prediction for corroded columns

Specimen	f'_c , MPa	V_{max} , kN	$\frac{V_{max}}{f'_c A_g}$	FS_{corr}	$Pit_{max}^{(1)}$, mm	$Pit_{max}^{(1)}/R_0$
U1	32.2	232.1	0.33	1.00	0.00	
C1	37.5	238.8	0.32	0.93	0.82	0.082
C2	32.0	193.2	0.28	0.88	1.36	0.136
C3	40.0	190.7	0.25	0.75	3.05	0.305
U2	38.1	302.5	0.40	1.00	0.00	
C4	30.0	254.4	0.38	0.94	0.62	0.062
C5	31.3	237.1	0.35	0.91	1.03	0.103

Notes: 1 mm = 0.04 in; 1 MPa = 0.145 ksi; 1 kN = 0.0225 kip.

can present a relatively good correlation with the reduction of their flexural strength. Therefore, a similar approach is used to predict the residual flexural strength of RC columns with corroded reinforcement in this paper. Due to the compressive concrete strength varied among test specimens, the flexural strength of test specimens was normalized as $V_{max}/A_g\sqrt{f'_c}$. Based on the regression analysis, the residual flexural strength of corroded columns is estimated as

$$FS_{corr} = -0.803(Pit_{max}^{(1)}/R_0) + 0.991 \quad (6)$$

where FS_{corr} is the residual flexural strength defined as the ratio of flexural strength of the corroded column to that of the uncorroded column; $Pit_{max}^{(1)}$ is the maximum pit depth of corroded longitudinal reinforcing bars estimated based on the experiment, as indicated in Table 2; and R_0 is the original radius of longitudinal reinforcement. Table 4 shows the calculation details of the residual flexural strength for corroded columns that failed in flexural and flexure-shear manners. Figure 15 demonstrates a good result of regression analysis, with an R^2 value of 0.848.

Proposed shear strength assessment

In this study, as mentioned previously, there were three corroded columns that failed in flexural-shear. Therefore, it is reasonable to propose the shear strength assessment for these corroded columns that considers the effects of reinforcement corrosion to their shear strength deterioration. Among various available shear strength models in literature, the shear strength model proposed in ASCE/SEI 41-13⁶ is being commonly used to predict the shear strength of RC columns that can incorporate the primary parameters of the column, including the shear span-to-depth ratio, applied axial force, cross-sectional dimensions, concrete strength, and displacement ductility demand. In this study, this shear strength model is used and modified to estimate the shear strength of corroded columns by considering the contribution reductions to the shear strength of transverse reinforcement and concrete due to corrosion. This approach has been also employed to investigate the influences of corroded transverse reinforcement on the shear strength of RC beams in literature.^{9,15} The original shear strength equation proposed in ASCE/SEI 41-13⁶ is given as

Fig. 15—Relationship between ratio of maximum corrosion pit depth to original radius of longitudinal reinforcement and residual flexural strength of corroded test specimens.

$$V_n = kV_0 = k \left[\frac{A_v f_{yt} d}{s} + \lambda \left(\frac{0.5 \sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \right] \quad (7)$$

$$k = \begin{cases} 1, \mu_\delta \leq 2.0 \\ 1.15 - 0.075\mu_\delta, 2.0 < \mu_\delta < 6.0 \\ 0.7, \mu_\delta \geq 6.0 \end{cases} \quad (8)$$

where k is a parameter to consider the reduction of shear strength with the increase of displacement ductility μ_δ ; a/d is the shear span-depth ratio; A_v is the total area of transverse reinforcement within the spacing s ; d is the effective depth that can be assumed as $d = 0.8h$, where h is the depth of the column cross section; A_g is the gross cross-sectional area of the column; $\lambda = 0.75$ for lightweight aggregate concrete and 1.0 for normalweight aggregate concrete; and P is the applied axial force. The contribution reductions of transverse reinforcement and concrete due to corrosion to the shear strength estimation of corroded columns are presented in the following section.

Transverse reinforcement contribution—Generally, corrosion of reinforcement reduces the mechanical properties and diameter of reinforcement, resulting in the reduction of total cross-sectional area A_v and yield strength of transverse reinforcement, f_{yt} . Therefore, a modification of cross-sectional area A_{vc} and yield strength f_{yt}^c for transverse reinforcement was made in this proposed shear strength assessment as the following equations

$$V_s^{pro} = \frac{A_{vc} f_{yt}^c d}{s} \quad (9)$$

$$A_{vc} = (1 - X_{corr}) A_v \quad (10)$$

$$f_{yt}^c = (1 - \alpha_s X_{corr}) f_{yt} \quad (11)$$

where X_{corr} is the corrosion level of transverse reinforcement. In this study, three considerations of cross-sectional area loss of transverse reinforcing bars were examined,

Table 5—Proposed shear strength assessment for corroded columns

Specimen	b , mm	d , mm	a , mm	A_{core} , mm ²	A_{cover} , mm ²	f'_c , MPa	ζ	X_{corr} , %	f_{yr}^c , MPa	A_{vc} , mm ²	s , mm	P , kN	μ_δ	k	V_s^{pro} , kN	V_c^{pro} , kN	V_n^{pro} , kN
C3	350	280	890	90,000	32,500	40.0	0.73	25	262.5	122.4	50	490.0	3.81	0.86	155.41	123.28	278.7
	350	280	890	90,000	32,500	40.0	0.73	30.5	254.3	113.4	50	490.0	3.81	0.86	139.49	123.28	262.8
	350	280	890	90,000	32,500	40.0	0.73	62.3	206.6	61.5	50	490.0	3.81	0.86	61.47	123.28	184.7
C4	350	280	890	90,000	32,500	30.0	0.81	13.6	279.6	141.0	50	918.8	3.57	0.88	194.65	141.61	336.3
	350	280	890	90,000	32,500	30.0	0.81	22.6	266.1	126.3	50	918.8	3.57	0.88	165.95	141.61	307.6
	350	280	890	90,000	32,500	30.0	0.81	41.3	238.1	95.8	50	918.8	3.57	0.88	112.59	141.61	254.2
C5	350	280	890	90,000	32,500	31.3	0.82	15.5	276.8	137.9	50	958.6	3.61	0.88	187.85	145.42	333.3
	350	280	890	90,000	32,500	31.3	0.82	29.5	255.8	115.0	50	958.6	3.61	0.88	144.83	145.42	290.2
	350	280	890	90,000	32,500	31.3	0.82	59.2	211.2	66.6	50	958.6	3.61	0.88	69.22	145.42	214.6

Notes: 1 mm = 0.04 in.; 1 mm² = 0.0016 in.²; 1 MPa = 0.145 ksi; 1 kN = 0.0225 kip.

including corrosion levels in terms of mass loss $X_{corr}^{(1)}$, average residual cross-sectional area $X_{corr}^{(2)}$, and minimum residual cross-sectional area $X_{corr}^{(3)}$, as defined from Eq. (1) to (3), respectively; α_s is the yield strength reduction factor for corroded reinforcement, taken as 0.005.¹⁶ Table 5 shows the calculation details of the shear strength due to transverse reinforcement contribution for corroded columns with three evaluative methods of cross-sectional area loss of transverse reinforcing bars.

Concrete contribution—Reinforcement corrosion results in the cracking of cover concrete, which reduces the contribution of concrete to the shear strength of RC columns. This adverse effect can be simulated by reducing the compressive concrete strength of cover concrete to $\zeta f'_c$ in which the softening coefficient ζ can be estimated as follows¹⁷

$$\zeta = \frac{0.9}{\sqrt{1 + 600\epsilon_r}} \quad (12)$$

$$\epsilon_r = W_{cr}/p_0 \quad (13)$$

where ϵ_r is the tensile strain induced by the cracks of cover concrete; W_{cr} is the total crack width of the specimen, which is defined in Eq. (4) and can be measured in the experiment and real engineering; and p_0 is the perimeter of the column section. As shown in Eq. (14), which is modified from the shear strength equation proposed in ASCE/SEI 41-13,⁶ the contribution from concrete to the shear strength of corroded columns is estimated by remaining the compressive concrete strength f'_c for the core concrete area A_{core} and using the reduction of compressive concrete strength of $\zeta f'_c$ for the cover concrete area A_{cover} as well as taking the effective depth $d = 0.8h$. Finally, the shear strength of corroded columns is estimated by summing the contributions to the shear strength of transverse reinforcement and concrete, as indicated in Eq. (15)

$$V_c^{pro} = \left(\frac{0.5\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{0.5\sqrt{f'_c}(A_{core} + \sqrt{\zeta}A_{cover})}} \right) 0.8(A_{core} + \sqrt{\zeta}A_{cover}) \quad (14)$$

$$V_n^{pro} = V_s^{pro} + V_c^{pro} \quad (15)$$

The calculation details of the shear strength provided by concrete for corroded columns are indicated in Table 5. Comparisons of the shear strength between the experimental results and proposed assessment for corroded columns are shown in Table 6. As indicated, compared to the experimental results, the shear strength prediction using the corrosion mass loss and average residual cross-sectional area was not conservative because the average ratios of the shear strength calculated based on the proposed shear strength assessment to the test results are 1.40 and 1.27, and their standard deviations of 0.071 and 0.095, respectively. However, the shear strength prediction using the minimum residual cross-sectional area of transverse reinforcement showed a good correlation with the test results; the average ratio of the shear strength estimated based on the proposed shear strength assessment to the test results is 0.96 and its standard deviation is 0.046.

CONCLUSIONS

In this study, the influence of reinforcement corrosion on the seismic performance of RC columns is experimentally and analytically investigated. The key parameters investigated and quantified include the corrosion levels of reinforcement and axial load ratios. Some important findings can be concluded from the experimental and analytical studies:

1. Reinforcement corrosion strongly affects the seismic performance of RC columns, including the hysteretic response, crack pattern, yield displacement, lateral load resistance, displacement ductility, drift capacity, and energy dissipation capacity, even more so when they are highly corroded and subjected to high axial load.

2. Three failure modes are observed in which two uncorroded columns and two corroded columns subjected to low corrosion level and axial load fail in typical flexural manner. The second failure mode is flexural-shear failure, which is directly caused by the formation and propagation of diagonal shear cracks in the plastic hinge regions of the columns. This failure is found in three corroded columns subjected to either high corrosion levels or high axial load. The third failure mode is sudden axial failure, which occurred in a corroded column subjected to high corrosion level combined with high axial load, resulting in the explosion of concrete and reinforcement at the top end of the column. This failure is

Table 6—Comparisons of shear strength of corroded columns using different evaluative methods of corrosion level of transverse reinforcement

Specimen	V_n^{exp} , kN	$X_{corr}^{(1)}$, %	$V_n^{pro(1)}$, kN	$V_n^{pro(1)}/V_n^{exp}$	$X_{corr}^{(2)}$, %	$V_n^{pro(2)}$, kN	$V_n^{pro(2)}/V_n^{exp}$	$X_{corr}^{(3)}$, %	$V_n^{pro(3)}$, kN	$V_n^{pro(3)}/V_n^{exp}$
C3	190.7	25.0	278.7	1.46	30.5	262.8	1.38	62.3	184.7	0.97
C4	254.4	13.6	336.3	1.32	22.6	307.6	1.21	41.3	254.2	1.00
C5	237.1	15.5	333.3	1.41	29.5	290.2	1.22	59.2	214.6	0.91
Average				1.40			1.27			0.96
COV				0.071			0.095			0.046

Note: 1 kN = 0.0225 kip.

mainly caused by the loss of confinement effectiveness for confined concrete and buckling resistance of longitudinal reinforcement due to the fracture of transverse reinforcing bars induced by extremely severe corrosion.

3. The test results reveal that the increase of corrosion levels results in the significant deterioration of the lateral load and drift capacities of RC columns. In addition, the adverse effect of reinforcement corrosion on drift capacity at failure is more serious than that on the lateral load capacity. Furthermore, the displacement ductility of corroded columns is also found to be reduced, corresponding to the increase of axial force ratios and corrosion levels of reinforcement. The test results also show that the energy dissipation capacity of corroded columns strongly depends on the corrosion levels of reinforcement and applied axial load ratios. Specifically, the energy dissipation capacity decreases with the increase of corrosion levels and applied axial force ratios.

4. This study suggests an empirical equation to predict the residual flexural strength of corroded columns based on the experimental results, which is a function of the maximum pit depth of corroded longitudinal reinforcing bars, showing good agreement with the experimental results. Furthermore, a method to predict shear strength of corroded columns that failed in flexural-shear and axial failure is also developed based on modifying the shear strength model in ASCE/SEI 41-13, providing good and reliable correlation with the test results. It will be better if more experiments are conducted to verify these proposed methods to predict the residual flexural strength and shear strength of RC columns with corroded reinforcement.

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