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Seismic Performance Assessment of L-Shaped Reinforced Concrete Columns

by Thanh Phuong Pham and Bing Li

This paper presents an experimental investigation carried out on 10 reinforced concrete (RC) columns with L-shaped sections subjected to simulated seismic loadings. The primary objective of the study was to assess the suitability of using L-shaped RC columns in regions of low-to-moderate seismic risk. The variables of interest included the axial load, loading direction, and aspect ratio. The specimen performances were analyzed and discussed in terms of crack patterns, hysteretic response, curvature distribution, displacement decomposition, strain profile, and energy dissipation capacity. Results highlighted that the L-shaped columns possessed good seismic resistance, that failure mechanism of L-shaped columns was mainly governed by vertical splitting cracks, and that shear strength of L-shaped columns was not significantly affected by the level of applied axial force and the direction of horizontal force while initial stiffness was considerably affected by aspect ratio. Further analysis showed that ignoring the contribution of flanges, the shear equation (special provisions for walls) in ACI 318-11 yields good estimations of shear strength for the cases of L-shaped columns.

Keywords: L-shaped columns; reinforced concrete; seismic; shear strength.

INTRODUCTION

Irregularly shaped columns are required in some structural designs as at the corner of frame structures or at the enclosure of elevator shafts. Particularly, L-shaped columns are commonly used in existing buildings, especially in Singapore; an example is shown in Fig. 1. Unlike conventional rectangular or circular columns, which have been extensively studied over the years, the load and deformation behavior of L-shaped reinforced concrete columns are rarely available in literature. Marin¹ presented design aids for L-shaped columns using numerical investigations, Ramamurthy and Khan² suggested two methods of designs for such columns including the “failure surface” and the “equivalent” rectangular column, while Hsu³ presented theoretical and experimental investigation for L-shaped columns. Most of the work carried out by the aforementioned researchers focused on static or sectional analysis of such columns. Therefore, the seismic behavior of L-shaped reinforced concrete (RC) columns under combined cyclic bending and axial compression is still not well understood. Consequently, in design practice, engineers generally assume that the L-shaped columns respond as two separate, perpendicular, barbell walls. However, the validity of this assumption has not been well validated. To ascertain the validity of this assumption, seismic responses of the L-shaped columns are studied. The results can also be used to establish the influences of different column sections on the overall performance.

(a) Typical L-shaped column

(b) Typical L-shaped column

Fig. 1—Typical residential building in Singapore.

This paper aims to provide in-depth understanding of the seismic behavior of existing RC columns with L-shaped section. Ten half-scale RC columns were tested under a combination of a constant axial load and a cyclic loading. The test results were examined in terms of crack patterns, hysteretic response, curvature distribution, displacement decomposition, strain profile, and energy dissipation capacity. The results were then compared with predictions based on shear strength from current codes for RC flanged walls. From the above discussion, the following goals were identified:

- Assess the failure mechanism of L-shaped columns, especially to ascertain if the L-shaped columns are susceptible to shear diagonal cracking, which normally occurs in nonductile square columns;^{4,5}
- Procure experimental evidence for strength, stiffness degradation, and deformation capacity of L-shaped columns, especially when subjected to different loading directions; and
- Assess the possibility of applying shear strength model for RC flanged walls to L-shaped columns.

The findings in this study provide useful information for both structural design and assessment.

RESEARCH SIGNIFICANCE

Reinforced concrete columns having L-shaped sections are commonly built in regions of low-to-moderate seismic risk.

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Table 1—Summary of test specimens

Group	Specimen	Section	L, mm	α , deg	$P/f'_c A_g$	A_s , mm ²	A_v , MPa	s, mm	f'_c , MPa	V_y , kN	V_u , kN	$I, \times 10^9$ mm ⁴
		$l_w \times l_w \times t$										
1	S1	430 x 430 x 170	1700	0	0.2	2411.5	56.5	100	29.8	155.9	186.4	1.80
	S2				0.35	2411.5	56.5	100	29.2	199.2	232.7	1.80
	S3			45	0.2	2411.5	56.5	100	29.2	219.7	267.1	2.52
	S4				0.35	2411.5	56.5	100	29.4	261.0	323.7	2.52
	S5			-45	0.2	2411.5	56.5	100	29.1	121.7	181.2	1.09
	S6				0.35	2411.5	56.5	100	29.0	144.0	183.9	1.09
2	S13	430 x 430 x 170	1200	0	0.2	2411.5	56.5	150	27.0	216.3	257.3	1.80
	S14				0.35	2411.5	56.5	150	27.4	272.3	316.7	1.80
	S15			45	0.2	2411.5	56.5	150	26.8	300.5	366.7	2.52
	S16				0.35	2411.5	56.5	150	27.2	357.0	436.4	2.52

Notes: 1 mm = 0.04 in.; 1 kN = 0.04 in.; 1 mm⁴ = 2.31×10^{-6} in.⁴; 1 MPa = 145 psi.

Fig. 2—Detail of specimens. (Note: All dimensions are in mm; 1 mm = 0.04 in.)

However, understanding of the cyclic behavior of L-shaped RC columns is very limited. This paper presents a comprehensive experimental investigation to provide insight into the seismic response of this type of RC columns. Results are helpful for both structural design and assessment purposes.

EXPERIMENTAL PROGRAM

Specimen details

Ten half-scale model columns were designed for the tests with combined axial load and cyclic lateral, as shown in Fig. 2

and Table 1. The columns geometries, details, and materials were selected to be representative of columns in existing buildings. Column section with equal flange length and the flange length-to-width ratio between 2 to 3 are typically found in existing buildings in Singapore. The test columns were divided into two groups with clear height of 1700 or 1200 mm (66.93 and 47.24 in.). All specimens had the same cross-sectional dimensions of 430 x 430 x 170 mm (27.24 x 27.24 x 10.77 in.) and a longitudinal reinforcement ratio of 2.1% with 12T16 (16 mm). Transverse reinforcement 6 mm

Fig. 3—Experimental setup.

(0.24 in.) in diameter was spaced at 100 mm (3.94 in.) for columns in Group 1 (1700 mm [67 in.] height) and at 150 mm (5.90 in.) for columns in Group 2 (1200 mm [47.2 in.] height). Concrete cover to surface of longitudinal reinforcement was 25 mm (0.98 in.) for all specimens. The longitudinal and transverse reinforcement were characterized by a yield strength of 465 and 467 MPa (67.4 and 67.7 ksi), respectively. It should be noted that, in many existing buildings, there was no provision available for ductile design; therefore, transverse reinforcement was detailed typically without C-links, as shown in Fig. 2. The shear force V_u corresponding to the development of the theoretical flexural strength of columns was estimated using the material properties obtained through tests and in accordance with the recommendations provided by ACI 318.⁶ The results are tabulated in Table 1.

Test setup

The test specimens were attached to a loading apparatus, as shown in Fig. 3, which enabled the columns to be displaced under double-curvature bending. Two vertical actuators were used to control the axial force as well as to maintain zero rotation at the top of the specimen. A horizontal actuator was used to produce cyclic shear force to the top end of specimens. All actuators were pinned at both ends to allow for rotation during the test. The base of the specimens was fixed to a strong floor by four post-tensioned bolts.

Loading procedure

The axial load was slowly applied until the designated level. During each test, the column axial load was maintained by manually adjusting the vertical actuators after each loading step. The lateral load was applied cyclically through the horizontal actuator in a quasi-static fashion, as shown in Fig. 3.

Instrumentation

The test specimens were extensively installed with measuring devices both internally and externally. Lateral displacement was measured by two horizontal linear variable differential transducers (LVDTs) mounted parallel to the horizontal actuator. Shear and flexural deformations were recorded based on the reading of LVDT sets mounted

Fig. 4—Deformation measurement.

along the height of the specimens, as shown in Fig. 4. Strain gauges were used to capture strains on both longitudinal and transverse reinforcements at critical locations; the arrangement of strain gauges is shown in Fig. 2.

EXPERIMENTAL RESULTS AND DISCUSSION

Results from the tests of 10 specimens are presented and discussed in this section. The hysteretic responses of all test specimens are depicted in Fig. 5. The response of test specimens were highlighted by four performance levels: theoretical flexural yielding (PL1), maximum shear force (PL2), shear failure (PL3—shear strength dropped by 20%) and axial failure (PL4—when columns were unable to sustain the applied axial force). Generally the hysteretic responses show pinching and reduction of stiffness as well as reduction of shear capacity during repeated cyclic loadings. All six specimens in Group 1 experienced yielding of longitudinal reinforcement before attaining maximum shear force while none of the specimens in Group 2 developed flexural yielding. Details of cracking patterns, failure mode, hysteretic response (including yield displacement, initial stiffness, shear strength, and drift capacity), curvature distribution, displacement decomposition, and energy dissipation capacity are described in the following sections.

Cracking patterns and failure modes

Cracks on all surfaces of the specimens were marked at every peak displacement stage. Cracking patterns at maximum shear force and at axial failure are shown in Fig. 6 and 7. From the observed cracking patterns, two failure modes were identified.

The first failure mode occurred in Specimens S1, S2, S3, S4, S13, S14, S15, and S16 due to the development of vertical splitting cracks along longitudinal reinforcements. The typical illustration is shown in Fig. 8. In these specimens, flexural cracks perpendicular to column axis developed first in the regions close to the top and bottom ends when loaded to a drift ratio of 0.33%; 0.2% for columns under the principal loading direction and diagonal direction, respectively, regardless of aspect ratio. When increasing drift ratio by

Fig. 5—Hysteretic response of test specimens (PL1 is theoretical flexural yielding; PL2 is maximum shear force; PL3 is shear failure – shear strength dropped by 20%; and PL4 is axial failure – when columns were unable to sustain applied axial force). (Note: 1 mm = 0.04 in.)

0.1%, the flexural cracks in horizontal direction gradually changed to diagonal direction and extended to the column web due to the appearance of high shear stress. It was also observed that in columns with higher axial load, both the flexural and diagonal cracks decreased in size and length. The diagonal cracks initially formed at an angle of around 45 degrees to the column vertical axis, and increased with the applied shear force, eventually reaching a maximum angle of 60 degrees. Extensive diagonal cracks formed in the specimens tested in strongest direction (Specimens S3, S4, S15, and S16). However, these diagonal cracks did not

directly lead to the failure of the specimens; rather, the failure of these specimens can be mainly attributed by the vertical cracks along center longitudinal bars. These cracks occurred earlier in Specimens with higher axial force and were observed first at a drift ratio of 0.8% in S1 and S13; 0.67% in Specimens S2, S14, and S15; and 0.5% in Specimens S3, S4, and S16. At the end of the tests, concrete crushing occurred at both ends of test specimens and on the surface of vertical splitting cracks (refer to Fig. 7). Concrete spalling was also observed along the height of specimens. The failure mechanism depicted in Fig. 7 was similar to the failure pattern

Fig. 6—Crack patterns at maximum shear force of test specimens.

Fig. 7—Axial failure of test specimens.

Fig. 8—Typical failure mode.

in the tests of square and rectangular RC columns observed previously by the authors.⁷ This unique failure mechanism has also been explained by the authors in another work.⁸ In that work, Pham and Li⁸ proposed a failure mechanism for the splitting failure, as shown in Fig. 9. According to the proposed mechanism, the applied axial force P , applied bending moment M , and applied shear force V are carried out by a summation of internal forces: compression forces

C_1 and C_2 acting in both column ends; diagonal sliding shear stresses S_1 and S_2 of concrete in compression zones; vertical sliding shear stress S_3 in concrete along potential splitting plane; and tensile stress in transverse reinforcement T_s . By using the test results, Pham and Li⁸ proved that the splitting failure occurs when the vertical sliding shear stress S_3 reaches its ultimate sliding shear strength. The analytical

Table 2—Yield displacements and yield curvatures

Group	Specimen	Yield displacement, mm	$M_{First\ yield}$, kNm			Yield curvatures		
			Estimate	Test result	M_E/M_T	Estimate	Test result	K_{test}
1	S1	14.47	139.5	153.1	0.91	11.81	8.72	1.5
	S2	12.53	178.2	173.2	1.03	11.39	6.97	1.2
	S3	11.87	196.6	180.7	1.09	8.84	6.38	1.6
	S4	9.93	233.5	190.4	1.23	8.40	6.16	1.5
	S5	17.33	108.9	158.8	0.69	14.32	13.36	1.6
	S6	14.13	128.8	162.6	0.79	13.50	7.90	1.0
2	S13	7.47	136.6	109.0	1.25	11.41	—	—
	S14	5.07	172.0	114.3	1.50	11.15	—	—
	S15	7.27	189.8	123.3	1.54	8.62	—	—
	S16	4.73	225.5	125.4	1.80	8.15	—	—

Notes: 1 mm = 0.04 in.; 1 kN = 0.04 in.; 1 MPa = 145 psi.

Fig. 9—Failure mechanism.

results matched reasonably with the test observation and shear strength of the test columns.

The second failure mode was observed in specimens tested in weakest loading direction—S5 and S6. They failed due to the development of plastic hinges forming at both column ends and leading to crushing of concrete as well as buckling of longitudinal reinforcement. In these two specimens, flexural cracks developed at column ends from a drift ratio of 0.2% and 0.33% in S5 and S6, respectively. No sign of shear cracks was observed up to a drift ratio 1% in S5 and 0.8% in S6. Vertical cracks were also observed along the center bars of both flanges in loading to a drift of 1.67% in S5 and 1.43% in S6; however, these cracks did not propagate and had a negligible contribution to the failure.

Hysteretic response

Hysteretic response of test specimens (Fig. 5) was assessed by analyzing yield displacement, initial stiffness, maximum shear force, and drift capacity.

Yield displacement—Different definitions of yield displacement can be found in the literature from both experimental and analytical studies. In this study, yield displacement is defined as the displacement corresponding to the intersection point of the following two lines in the envelope curve: Line 1 passing through the origin and $0.75V_{max}$, and Line 2 passing through V_{max} and parallel to the x-axis. Yield displacements from the tests listed in Table 2 were taken as the mean values from pushing and pulling directions. The results show that yield displacement decreased with an increase of axial load or changing loading direction from weak direction (–45 degrees) to strong direction (45 degrees). In Group 2 (short columns), yield displace-

Fig. 10—Effects of test parameters on initial stiffness and shear strength.

ments were not governed by loading direction but significantly affected by axial load. When the ratio of axial force increased from 0.2 to 0.35, yield displacement reduced by almost 50% for columns in Group 2 while the reduction was only by 15 to 23% for columns in Group 1.

Initial stiffness—As a crucial parameter in the displacement-based design, initial stiffness is normally determined based on a point obtained from the measured force-displacement envelope with a shear force that is equal to the force at first yield. The shear force at first yield is defined as either the first yield that occurs within the longitudinal reinforcement or when the maximum compressive strain of the concrete attains a value of 0.002 at any critical section of the column. However, this definition would not be applicable to columns in this investigation, as shear strength does not substantially exceed the theoretical yield force. Therefore, the initial stiffness is defined based on a point on the measured force-displacement envelope with the shear force equal to 80% of the obtained maximum shear force.⁴

Figure 10 shows the effects of axial load, aspect ratio, and loading direction on initial stiffness. In the first group, when the applied axial force ratio increased from 0.2 to 0.35, the initial stiffness increased by 13%, 26%, and 31% for the columns tested under loading directions of 0, 45, and –45 degrees, respectively. Under the same axial load ratio of 0.2, the initial stiffness was increased by 37% and 18% when loading direction was changed from –45 to 0 degrees and from 0 to 45 degrees, respectively. The changes were 18% and 31% when the applied axial load ratio was 0.35. It is noted that the moment of inertia (I , as shown in Table 1) of the strongest direction (45 degrees) is 40% and 130% higher than that of 0- and 45-degree direction, respectively.

In the second group (the short columns), the changes in initial stiffness were not significantly noticeable when the loading direction was changed. There was only an 8.4% difference in initial stiffness between S14/S16 and 14.4% between S13/S15. However, the initial stiffness was appreciably increased when the applied axial load was increased. The increment recorded was approximately 55% when axial load ratio was increase from 0.2 to 0.35 (S13 versus S14; S15 versus S16).

Tran and Li⁴ found out that initial stiffness of RC columns is not significantly affected by amount of transverse reinforcement. Therefore, a direct comparison can be carried

out between the stiffness of the two specimen groups with different transverse reinforcement ratios. As shown in Fig. 10, the initial stiffness was increased by an average of 2.1 times when the aspect ratio reduced from 2.0 to 1.4. The largest increment of 2.8 times was found in the comparison between S2 and S14 followed by S4 versus S16. The smallest increment was that of the pair S3 versus S15 where the initial stiffness of S15 was 66% higher than that of S3.

It can be concluded that initial stiffness is less sensitive to loading direction of short L-shaped columns than the longer ones. Initial stiffness is considerably susceptible to the axial load of short L-shaped columns rather than the longer ones. Nevertheless, the aspect ratio was found to be the most critical parameter influencing the initial stiffness of L-shaped columns.

Shear strength—Shear strength was normalized by dividing the attained maximum shear force (from hysteretic response) to the cross-sectional area of specimen and square root of compressive concrete strength ($V_{max}/A_g\sqrt{f'_c}$). The results in Fig. 10 show that the normalized shear strength of all test specimens was varied within a short range from 0.36 to 0.44. The maximum shear force of L-shaped columns was not significantly affected by applied axial force levels. For the first group, when the axial force ratio increased by 75% from 0.2 to 0.35, the shear strength was increased only by 11%, 4%, and 0.1% for columns in directions of 0, 45, and –45 degrees, respectively. The increments were also around 3.9% and 1.9% for the group of short columns. This observation can be explained by the high value of the depth-to-width ratio in L-shaped column sections in which the mechanism of axial force influencing the shear strength through compression diagonal strut might not mobilize.

Another observation showed that, although the section is not symmetrical, the shear strength was surprisingly not remarkably affected by the loading direction. Highest change in shear strength of 19.8% was found in a specimen of the first group when tested under the applied axial ratio of 0.2 and the loading direction was changed from 0 to 45 degrees (S1 versus S3). The changes in short columns were 10.3% and 13.5% for the columns tested under axial load of 0.2 and 0.35, respectively. Nonetheless, loading direction seems to be sensitive to the shear strength rather than the applied axial force.

Drift capacities—Drift capacities—drift ratios recorded at each the performance level (as defined previously)—are shown in the hysteretic responses of each specimen in

Fig. 11—Effects of test parameters on drift capacity.

Fig. 5. The effects of the test parameters on the drift capacities of L-shaped columns are depicted in Fig. 11. The results show that drift ratios at PL2, PL3, and PL4 of all specimens reduced with an increase of axial force or the change of loading direction from weaker direction to stronger direction. With an increment in the initial stiffness, the specimen behaves in a more brittle manner, which results in a lower drift capacity. The results also show that when the axial force ratio increased from 0.2 to 0.35, the most significant change in drift capacity was observed in the specimen with clear height of 1.7 m (5.6 ft) and tested in the strongest loading directions. From S3 to S4, the drift ratio was reduced by 24.4%, 75%, and 82.2% for the points at maximum shear force, shear failure, and axial failure, respectively. With respect to the effect of loading direction, the change in drift capacities was most notably observed in S2 and S4, which tested under the same axial force ratio of 0.35. The change in loading direction by 45 degrees from S2 to S4 resulted in the reductions in drift capacities of 25%, 68%, and 85% for the performance levels PL2, PL3, and PL4, respectively.

Curvature distribution

Experimental curvatures of test specimens reported herein were calculated as

$$\phi = \frac{\Delta_t - \Delta_c}{h_g l_g} \quad (1)$$

where Δ_t and Δ_c are the elongations of LVDTs on the tension and compression sides of the member, respectively; l_g is the height of column portion at which one set of two LVDTs instrumented; and h_g is the center-to-center distance between the two LVDTs. Yield curvatures corresponding to

yield displacements (defined previously) and curvatures at maximum shear force attained for both push and pull directions are shown in Fig. 12.

It is shown that for Group 1, curvatures varied linearly along 6% to 94% column height for both yield curvature and curvature at maximum shear force. It is observed that the main source of the flexural displacement was the deformation from base-rotation, which accounted for 55% to 72% of the total flexural deformation. These percentages remained approximately similar at PL1 and PL2. Yield curvatures were slightly reduced with increasing applied axial force except for S5 and S6, when loading in the weakest direction.

The theoretical yield curvature is derived from the first yield base moment and nominal base moment

$$\phi_y = \frac{\phi'_y M_n}{M_y} \quad (2)$$

In this study, the theoretical first yield base moment M_y is calculated as the base moment at which the reinforcement reaches its yield strain or the compressive strain of the concrete attains 0.002. The variable ϕ'_y is the curvature corresponding to the first yield base moment. The nominal base moment M_n was taken when strain of extreme fiber reached 0.004 or when strain in the longitude reinforcement reached 0.015, whichever occurs first. It is worth noting that the estimated moment at first yield M_y in the Group 1 matches well with the moment at first yield from experiment defined based on $0.75V_{max}$. Therefore, comparisons of yield curvatures between theoretical and experimental results could be carried out. The comparisons in Table 2 show that the yield curvatures were overestimated by this method. The average difference was 42% with the coefficient of variation

Fig. 12—Curvature distribution. (Note: 1 mm = 0.04 in.)

Fig. 13—Displacement decomposition.

of 28%. Nevertheless, both experimental results and theoretical results agree that yield curvature reduced with increased axial force and changing horizontal loading from the strong direction to weak direction.

Priestley and Kowalsky⁹ proposed an equation to predict the yield curvature of structural walls by its wall length

$$\phi_y = K \frac{\epsilon_{sy}}{l_w} \quad (3)$$

where ϵ_{sy} is yield strain of longitudinal reinforcement; l_w is the web length of columns section (l_w taken as 430 mm, $430\sqrt{2}$ mm, and $430/\sqrt{2}$ mm for columns tested under loading direction of 0, 45, and -45 degrees, respectively). The calculated K values for L-shaped columns based on experimental results of yield curvatures of columns in Group 1 are shown in Table 2. It is shown that the K was approximately 1.5 when the applied axial load ratio was 0.2 regardless of loading directions. However, different K

values resulted from different loading directions when the applied axial force ratio was 0.35 with the highest K value of 1.50 recorded in the strongest loading direction (45 degrees), and the lowest K value of 0.96 was found in the weakest direction (-45 degrees).

Displacement composition

LVDTs attached to the faces of the specimens provide data that can be used to calculate the contribution of flexural, slip, and shear deformation to the total horizontal displacement. Flexural and slip deformation were calculated based on average curvatures as presented previously. Shear deformation was calculated as the change in length of the diagonal LVDTs divided by the sine of the angle between the diagonal and column axis. Figure 13 presents the composition of each deformation component for test specimens: the top part is the contribution of slip deformation, the middle part is the contribution of shear deformation, and the lower part represents the contribution of flexural deformation. The values do not add up to exactly 100% of the total

Table 3—Drift ratio at yielding of reinforcements from strain gauge measurement

Group	Specimen	First yield of reinforcing bar			First yield of stirrup		
		+ δ , %	− δ , %	+ $\delta_{(mean)}$	+ δ , %	− δ , %	+ $\delta_{(mean)}$
1	S1	1.26	−0.67	0.97	0.86	−0.88	0.87
	S2	0.74	−0.99	0.87	1.69	−1.47	1.58
	S3	1.43	−1.01	1.22	1.04	−1.42	1.23
	S4	1.24	−0.64	0.94	1.24	−1.34	1.29
	S5	0.95	−0.64	0.80	3.55	−1.39	2.47
	S6	0.68	−0.62	0.65	2.25	−2.92	2.59
2	S13	0.90	−0.98	0.94	1.99	−1.40	1.70
	S14	0.61	−0.83	0.72	2.78	−2.90	2.84
	S15	—	−1.04	1.04	1.24	—	1.24
	S16	0.90	−0.70	0.80	1.45	−2.59	2.02

displacement due to experimental error. The results show that in loading to $0.75V_{max}$, the contribution of slip to total displacement was accounted for 20 to 70% with average of 44%. This contribution was reduced by an average of 10% when loading to V_{max} in all specimens except S5 and S6. The contribution of shear deformation was found to be largest in specimens tested in the principal direction, followed by the directions of 45 degrees. Shear deformation was relatively small in specimens tested in the direction of −45 degrees (the weakest direction) with the ratio of 14% and 10% total deformation in Specimens S5 and S6, respectively, when maximum shear force was attained (PL2). The results also indicate that the contribution of shear deformation was slightly higher in the columns with lower applied axial force.

Strain in reinforcement

Strain gauges were attached to both longitudinal and transverse reinforcements. The strain gauge reading generally indicated regions of highest distress along the specimen and coincided with stress as evident from the cracking patterns. Highest strain was recorded in outermost longitudinal reinforcement and transverse reinforcement at a distance of two-tie spacing from the base. Yielding of longitudinal reinforcement and transverse reinforcements was recorded and tabulated in Table 3. Generally, all specimens attained yielding of longitudinal reinforcement when loaded to a drift ratio in the range of 0.5 to 1%. Although theoretically there should be no flexural yielding in short columns, the test results showed that all specimens, including short columns in Group 2, experienced yielding of longitudinal prior to maximum shear force was attained. This is possibly due to the strain penetration resulting from the slippage of the longitudinal bars at the column-foundation interface. The results from Table 3 also indicate that with increasing applied axial force, the longitudinal bars will reach yield strain earlier; however, a reverse affect is observed in transverse reinforcement. This is because when the applied axial force is increased, the combined compression stress (by bending moment and axial force) is quickly increased, causing the longitudinal reinforcement to reach compression

Table 4—Energy dissipated at each performance level

Group	Specimen	Energy (kNmm) dissipated at			
		PL1	PL2	PL3	PL4
1	S1	4.5	22.2	64.5	93.3
	S2	4.1	20.7	60.0	76.0
	S3	6.2	22.8	67.8	89.0
	S4	4.5	20.5	40.1	55.0
	S5	5.4	25.7	104.5	114.0
	S6	5.1	25.1	85.6	96.0
2	S13	2.7	15.5	46.2	78.0
	S14	2.6	8.4	38.9	70.7
	S15	3.4	17.0	36.7	83.0
	S16	2.4	9.4	43.0	70.8

yielding earlier. Similarly, columns loaded in the weaker direction saw yielding of longitudinal bars earlier and reached yielding of tie later than those loaded in stronger direction.

Energy dissipation capacity

Physically, the total energy dissipated during the hysteretic response of RC member was calculated as the area enclosed by hysteretic cycles. Results for each performance level of all test specimens are presented in Table 4. It was expected that when the specimens respond in a more brittle manner, the energy dissipation capacity of the columns will be reduced. Therefore, the energy dissipation capacity of specimens was higher in specimens with lower applied axial force or higher aspect ratio. However, the results show that when shear strength was attained (PL2), there were only slight differences between energy dissipated by L-shaped columns under axial force ratio of 0.35 and with those columns under axial force of 0.2 in Group 1. The reductions were 7.6%, 11.3%, and 2.4% for S1 to S2, S3 to S4, and S5 to S6, respectively. The short columns, on the contrary, were substantially sensitive with the increment of applied axial force. The reductions by 84.5% and 80.8% were found in the comparisons between S13 with S14 and S15 with S16, respectively.

In both groups, at PL1 and PL2, the loading direction did not show any significant effect on the energy dissipated of all specimens. Shear failure and axial failure only occurred after dissipating amount of energy equal to 2 to 4.6 times and 2.68 to 8.41 times the amount dissipated at maximum shear force.

SHEAR STRENGTH EVALUATION

The effect of flanges on the shear strength of L-shaped RC columns has not been well studied and is usually ignored in design practice, which is similar to the observations in RC flanged walls. In this section, shear strength models, which were originally proposed for walls, will be assessed and compared with the test results of L-shaped columns to evaluate the possibility in the application of these models for L-shaped columns.

Three equations based on the procedures provided in Chapter 21 of ACI 318-11,⁶ Chapter 11 of ACI 318-11, and

ASCE 43¹⁰ are used herein to predict the peak shear resistance of the test L-shaped columns. ACI 318-11 provides two semi-empirical equations, both based on the modified truss analogy approach, to evaluate the strength of squat reinforced concrete walls. The equation in Section 21.9 of ACI 318-11 (Special reinforced concrete structural walls and coupling beams) is intended for seismic design. The equation in Section 11.9 (Special provisions for walls), is to be used for general design.

Equation (4) is from Section 21.9 of ACI 318-11, thereafter, namely, Eq. (1)

$$V_1 = (\alpha_c \sqrt{f'_c} + \rho_t f_{yt}) 0.8 A_w \leq 0.83 \sqrt{f'_c} A_w \quad (4)$$

where the coefficient α_c is 3.0 for $h_w/l_w \leq 1.5$, is 2.0 for $h_w/l_w \geq 2.0$, and varies linearly between 3.0 and 2.0 for h_w/l_w between 1.5 and 2.0.

Equation (1) imposes an upper limit of $0.83 \sqrt{f'_c}$ MPa ($10 \sqrt{f'_c}$ psi) on the ultimate shear stress—a limit intended to prevent diagonal compression failure. A lower limit of 0.25% is imposed on the horizontal and vertical wall reinforcement ratios. For walls with aspect ratios less than or equal to 2.0, Eq. (1) requires that the vertical reinforcement ratio be no less than the horizontal reinforcement ratio.

The procedure for evaluating the shear strength per Section 11.9 of ACI 318-11, thereafter namely Eq. (2), is given by the following four equations

$$V_2 = V_c + V_s \leq 0.83 \sqrt{f'_c} t_w d \quad (5)$$

$$V_c = 0.274 \sqrt{f'_c} t_w d_1 + \frac{P_u d}{4 l_w} \quad (6)$$

$$V_c = \left[0.05 \sqrt{f'_c} + l_w \left(0.104 \sqrt{f'_c} + \frac{0.2 P_u}{l_w t_w} \right) / \left(\frac{M_u}{V_u} - \frac{l_w}{2} \right) \right] t_w d \quad (7)$$

$$V_s = \frac{A_v f_{yt} d}{s} \quad (8)$$

The following conditions apply per Section 11.9.6 of ACI 318-11: the shear strength provided by concrete is taken as the lesser of the values provided by Eq. (6) and (7); Eq. (7) does not apply when $M_u/V_u - l_w/2 \leq 0$. ACI 318-11 Chapter 11 imposes an upper limit of $0.83 \sqrt{f'_c}$ MPa ($10 \sqrt{f'_c}$ psi) on the ultimate shear stress. The ratio of the minimum horizontal reinforcement is restricted to 0.25%; the ratio of minimum vertical reinforcement ratio is determined by

$$\rho_v = 0.0025 + 0.5 \left(2.5 - \frac{h_w}{l_w} \right) (\rho_t - 0.0025) \quad (9)$$

The equation described by Eq. 4-3 and 4-4 in ASCE 43-05³ to compute the shear strength of squat walls with barbell or flanges, thereafter namely Eq. (3). The equation is applicable for walls with the aspect ratios h_w/l_w of 2 or less and vertical

Table 5—Estimation of shear strength for L-shaped columns

Specimen	V_{test} , kN	Prediction, kN					
		V_1	V_1/V_{test}	V_2	V_2/V_{test}	V_3	V_3/V_{test}
S1	228.6	326.7	1.43	215.0	0.94	370.0	1.62
S2	254.2	326.7	1.29	262.9	1.03	392.1	1.54
S13	234.4	315.3	1.35	234.1	1.00	378.3	1.61
S14	244.7	315.3	1.29	318.8	1.30	378.3	1.55
S7	142.2	235.8	1.66	156.1	1.10	282.9	1.99
S8	161.2	237.3	1.47	135.2	0.84	284.8	1.77
Mean		1.41	—	1.04	—	1.68	—
COV		0.106	—	0.148	—	0.109	—

Note: 1 kN = 0.2248 kip.

and horizontal reinforcement ratios less than or equal to 1%. If the reinforcement ratios exceed 1%, the combined reinforcement ratio ρ_{se} (calculated using Eq. (12)) is limited to 1%. ASCE 43 imposes an upper limit of $1.66 \sqrt{f'_c}$ MPa ($20 \sqrt{f'_c}$ psi) on the ultimate shear stress.

$$V_3 = v_n d t_w \quad (10)$$

$$v_n = 0.689 \sqrt{f'_c} - 0.282 \sqrt{f'_c} \left(\frac{h_w}{l_w} - 0.5 \right) + \frac{P}{4 l_w t_w} + \rho_{se} f_y \leq 1.66 \sqrt{f'_c} \quad (11)$$

$$\rho_{se} = A \rho_v + B \rho_t \quad (12)$$

The shear strength prediction results for L-shaped columns tested in principal direction are shown in Table 5. In this table, shear strength of two other L-shaped columns with low concrete compressive strength ($f'_c = 15$ MPa) are also included for comparison (details and test parameters of these two specimens, S7 and S8, are similar to Specimens S1 and S2, respectively, except the transverse reinforcement spacing is 150 mm [5.9 in.]; the response of these two specimens is not presented herein). It is shown that the Eq. (2) from ACI 318-11 provides the best fit with the experimental results, with a mean of 1.04 and coefficient of variation of 14.8%. The comparison provides evidence that shear strength model in ACI 318-11 for walls could be used as a tool for shear strength assessment of L-shaped columns ignoring the contribution of concrete flanges. It should be noted that the flange would only effectively improve shear strength if it is in compression (as in the T-beam and L-beam). When a column is under double-curvature bending, the flanges of the column are under compressive stress at one column end and under tensile stress at the other end. In this case, the shear strength of the column will be governed by the weaker end (the column end with the flange is under tensile stress); therefore, the flanges do not significantly affect the seismic shear strength of L-shaped columns. Nevertheless, only a limited number of the test results from this study are used to investigate the applicability of the aforementioned three shear strength models for L-shaped columns. More experimental data with a wider range of variables are suggested to verify the findings from this study.

SUMMARY AND CONCLUSIONS

An experimental program was carried out on 10 reinforced concrete columns with L-shaped sections under simulated gravity and seismic loads. The obtained results were analyzed, discussed, and evaluated by the current assessment code for RC walls. The following conclusions are drawn:

1. All specimens responded with significant pinching and experienced yielding of longitudinal reinforcement before the maximum shear force was attained. Displacement can be primarily attributed to the slippage of longitudinal bars at the column-base interfaces. Shear deformation increased along with increasing applied horizontal load and was found to be largest in columns loaded in the principal direction, followed by columns in strongest direction, while columns loaded in weakest direction shear deformation only accounted for 10 to 14% of the total displacement. Two failure modes were observed whereas eight out of 10 specimens failed by the development of the vertical splitting cracks along the longitudinal reinforcement prior to reaching shear strength. These cracks separated the columns vertically, causing crushing and spalling of concrete in the sliding surface. The second failure mode was caused by the development of plastic hinges, which led to the crushing of concrete and the buckling of reinforcing bars at column ends. This failure was observed in two specimens tested under weakest loading direction. There was no significant difference in the hysteretic response between the two failure modes.

2. Unlike square or rectangular columns, axial force did not significantly affect the shear strength of L-shaped columns even when the compressive stress was less than $0.35f'_c$. The loading direction also showed minimal effect on the shear strength of L-shaped columns, where the largest difference was 19.8% in Group 1 and 13.5% in Group 2, when loading direction was changed.

3. Compared to L-shaped columns in Group 1, initial stiffness of short L-shaped columns in Group 2 is less sensitive to the change of loading direction but is significantly affected by a change in applied axial load. Aspect ratio was found to be the most critical parameter influencing the initial stiffness of L-shaped columns.

4. Previous studies have found that axial compressive force does not significantly affect the nominal yield curvature. However, this finding was only validated for walls with axial force ratio less than 0.1. For L-shaped columns with an axial force ratio of 0.2 to 0.35, yield curvature varies inversely with applied axial force and also affected by the loading direction.

5. The shear strength model provided in Section 11.9 in ACI 318-11 for RC flanged walls yields a good prediction for shear strength of L-shaped columns by ignoring the contribution of flanges. The estimated results match well with the shear strength of L-shaped columns tested in principal direction.

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NOTATION

A_g	=	cross-sectional area
A_v	=	total transverse reinforcement area within spacing
A_w	=	shear area of section—that is, area of web
b	=	width of column section
d	=	distance from extreme compression fiber to centroid of tension reinforcement
I	=	initial stiffness
E	=	cumulative energy dissipation
f'_c	=	compressive strength of concrete
f_y	=	yield strength of longitudinal reinforcement
f_{yt}	=	yield strength of transverse reinforcement
L	=	clear height of column
l_w	=	equivalent column section height
h	=	height of column section
h_w	=	web height
M	=	applied bending
M_n	=	nominal moment capacity
M_u	=	ultimate moment capacity
M_y	=	moment at first yield
P	=	applied axial load
s	=	spacing of transverse reinforcement
t_w	=	web thickness
V	=	applied shear force
V_c	=	contribution to shear resistance of concrete mechanism
V_{max}	=	maximum of applied horizontal force
V_n	=	nominal shear strength of columns
V_s	=	contribution to shear resistance of steel truss mechanism
V_u	=	shear force corresponding to development of theoretical flexural strength of columns
α_c	=	coefficient defining relative contribution of concrete strength to nominal shear strength
ϵ_{sy}	=	yield strain of longitudinal reinforcement
ϕ	=	curvature at applied moment
ϕ_y	=	theoretical yield curvature
ϕ_y'	=	curvature at first yield
ρ_t	=	transverse reinforcement ratio
ρ_v	=	vertical reinforcement ratio

REFERENCES

- Marin, J., "Design Aids for L-Shaped Reinforced Concrete Columns," *ACI Journal Proceedings*, V. 76, No. 11, Nov. 1979, pp. 1197-1216.
- Ramamurthy, L. N., and Khan, T. A. H., "L-Shaped Column Design for Biaxial Eccentricity," *Journal of Structural Engineering*, ASCE, V. 109, No. 8, 1983, pp. 1903-1917. doi: 10.1061/(ASCE)0733-9445(1983)109:8(1903)
- Hsu, C.-T. T., "Biaxial Loaded L-Shaped Reinforced Concrete Columns," *Journal of Structural Engineering*, ASCE, V. 111, No. 12, 1985, pp. 2576-2595. doi: 10.1061/(ASCE)0733-9445(1985)111:12(2576)
- Tran, C. T. N., and Li, B., "Initial Stiffness of Reinforced Concrete Columns with Moderate Aspect Ratios," *Advances in Structural Engineering*, V. 15, No. 2, 2012, pp. 265-276. doi: 10.1260/1369-4332.15.2.265
- Pham, T. P., and Li, B., "Seismic Behavior of RC Columns under Different Loading Directions," *ACI Structural Journal*, V. 110, No. 5, Sept.-Oct. 2013, pp. 833-843.
- ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary," American Concrete Institute, Farmington Hills, MI, 2011, 503 pp.
- Pham, T. P., and Li, B., "Seismic Performance of Reinforced Concrete Columns with Plain Longitudinal Reinforcing Bars," *ACI Structural Journal*, V. 111, No. 3, May-June 2014, pp. 561-572. doi: 10.14359/51686572
- Pham, T. P., and Li, B., "Splitting Failure of Reinforced Concrete Columns," *Journal of Structural Engineering*, ASCE, V. 140, No. 3, 2014, doi: 10.1061/(ASCE)ST.1943-541X.0000859
- Priestley, M. J. N., and Kowalsky, M. J., "Aspects of Drift and Ductility Capacity of Rectangular Cantilever Structural Walls," *Bulletin of the New Zealand National Society for Earthquake Engineering*, V. 31, No. 2, 1998, pp. 73-85.
- ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," American Society of Civil Engineers, Reston, VA, 2005, 98 pp.