Experimental and Analytical Studies on H-Shaped Reinforced Concrete Squat Walls

Ma, Jiaxing; Li, Bing

2018

Ma, J., & Li, B. (2018). Experimental and Analytical Studies on H-Shaped Reinforced Concrete Squat Walls. *ACI Structural Journal*, 115(2), 425-438.

<https://hdl.handle.net/10356/88753>

<https://doi.org/10.14359/51701144>

© 2018 American Concrete Institute. This paper was published in *ACI Structural Journal* and is made available as an electronic reprint (preprint) with permission of American Concrete Institute. The published version is available at: [<http://dx.doi.org/10.14359/51701144>]. One print or electronic copy may be made for personal use only. Systematic or multiple reproduction, distribution to multiple locations via electronic or other means, duplication of any material in this paper for a fee or for commercial purposes, or modification of the content of the paper is prohibited and is subject to penalties under law.

Downloaded on 26 Apr 2023 20:03:36 SGT

Title No. 115-S34

Experimental and Analytical Studies on H-Shaped Reinforced Concrete Squat Walls

by Jiaxing Ma and Bing Li

Reinforced concrete H-shaped squat walls are the primary lateral-load-carrying element in structures designed for protective purposes. To provide insight into their seismic responses, four H-shaped reinforced concrete (RC) squat walls were tested under lateral horizontal displacements and low levels of axial compression. Among them, two were imposed with lateral loading 45 degrees from the web to explore their behaviors under non-principal bending action. The seismic performance of specimens was discussed in terms of cracking patterns, failure mechanisms, hysteretic responses, deformation components, and strain profiles. To account for the vertical slip planes observed in HP5D0, an analytical model was developed, which revealed the deformation incompatibility reached its maximum at the height of $0.42h_w$. Based on cracking patterns, free body diagrams were also proposed to describe the force-transfer mechanism and to assess the peak shear strength of H-shaped squat walls. The proposed equations proved to be more accurate than those in ACI 318-14.

Keywords: H-shaped; reinforced concrete; seismic performance; shear strength; squat wall; vertical slip plane.

INTRODUCTION

Non-rectangular reinforced concrete (RC) squat walls are required in some structural designs, such as the corner of structures and elevator shafts. Particularly, H-shaped squat walls are widely employed in existing buildings as well as safety-related nuclear facilities¹ due to their significant strength and stiffness in both principal directions, which contributes to the resistance to biaxial bending action during earthquakes.

Unlike conventional rectangular walls that have been extensively studied in the past few decades, the seismic behavior of H-shaped RC squat walls is still less understood in the literature. Furthermore, the irregular configurations and shear-dominated characteristics tend to complicate the evaluation of their seismic performance. Scholars were also aware of this problem and conducted a number of studies in the relevant area.²⁻⁷ The first cyclic test on flanged squat walls was performed by Barda et al.³ They suggested that the vertical web reinforcement was effective as shear reinforcement in producing a more distributed cracking pattern and reducing crack widths. Kitada et al.⁸ later carried out an experimental program on two large-scale H-shaped squat walls, concluding that the existence of large flanges made it complex to develop models for the peak shear strength. Palermo and Vecchio⁹ also conducted cyclic tests on two similar H-shaped RC squat walls, finding out a premature failure at the top of one specimen and vertical slip planes on the web of the other specimen. The latter failure mechanism was quite different from the conventional diagonal compression failure usually seen in heavily reinforced squat walls, which required further

and deep analyses. Moreover, most past studies were only focused on walls subjected to lateral loading parallel to the web segment. However, in real earthquakes, the seismic wave can reach structures from any direction, which necessitates experiments studying the seismic performance of walls under various lateral loading directions.

This paper aims to provide an in-depth understanding of seismic behaviors of non-rectangular RC squat walls, especially H-shaped walls with a symmetric plane section. A total of four H-shaped specimens were tested under a combination of constant axial loads and cyclic lateral loading at Nanyang Technical University (NTU). The test results were thoroughly examined in terms of cracking patterns, failure mechanisms, hysteretic responses, deformation components, and strain profiles. Moreover, an analytical model was proposed to account for the unique failure mechanism and to develop equations for the peak shear strength of H-shaped squat walls.

RESEARCH SIGNIFICANCE

Despite the popularity and the importance of H-shaped squat walls, relevant studies are still limited, especially for their seismic behaviors under non-principal bending action. Thus, reversed cyclic tests on four H-shaped squat walls were conducted. An analytical model was developed to account for the vertical slip planes, which were also observed in past experiments but lacked deep research. Moreover, equations were proposed to better predict the peak shear strength of H-shaped squat walls.

TEST PROGRAM

Details of specimens

A total of four specimens named HP0D0, HP0D45, HP5D0, and HP5D45 were designed and constructed based on provisions of ACI 318-14.¹⁰ To represent squat walls in nuclear facilities and facilitate comparisons, these walls were designed almost half scale of the NUPEC specimens⁸ and the specimens tested by Palermo and Vecchio.⁹ As illustrated in Fig. 1, the web segment—1300 mm (51.18 in.) in length, 1000 mm (39.37 in.) in height, and 100 mm (3.94 in.) in thickness—was reinforced with D10 deformed bars spaced at 150 mm (5.91 in.) as longitudinal reinforcement and R6 round bars spaced at 80 mm (3.15 in.) as horizontal reinforcement. The two flanges were 1500 mm

ACI Structural Journal, V. 115, No. 2, March 2018.

MS No. S-2016-356.R, doi: 10.14359/51701144, was received June 1, 2017, and reviewed under Institute publication policies. Copyright © 2018, American Concrete Institute. All rights reserved, including the making of copies unless permission is obtained from the copyright proprietors. Pertinent discussion including author's closure, if any, will be published ten months from this journal's date if the discussion is received within four months of the paper's print publication.

Fig. 1—Specimen dimensions with reinforcement details. (Note: Dimensions in mm; 1 mm = 0.04 in.)

Table 1—Details of specimens

Specimen	h_w/l_w	A_g , m ² (in. ²)	f'_c , MPa (ksi)	f_y , MPa (ksi)		Web		Flange		$P/f'_c A_g$, %	θ , deg
				R6	D10	ρ_v , %	ρ_h , %	ρ_v , %	ρ_h , %		
HP0D0	0.67	0.43 (666.50)	35.9 (5.2)	335 (48.6)	554 (89.4)	0.97	0.71	1.26	0.71	0.48	0
HP0D45	0.67	0.43 (666.50)	33.8 (4.9)	335 (48.6)	554 (89.4)	0.97	0.71	1.26	0.71	0.51	45
HP5D0	0.67	0.43 (666.50)	35.9 (5.2)	335 (48.6)	554 (89.4)	0.97	0.71	1.26	0.71	5.01	0
HP5D45	0.67	0.43 (666.50)	33.8 (4.9)	335 (48.6)	554 (89.4)	0.97	0.71	1.26	0.71	5.32	45

Notes: h_w/l_w is aspect ratio; A_g is wall section area; f'_c is compressive strength of concrete; f_y is yield strength of reinforcement; ρ_v is longitudinal reinforcement ratio; ρ_h is horizontal reinforcement ratio; $P/f'_c A_g$ is axial load ratio; and θ is angle between lateral loading and web segment.

(59.06 in.) long, 1000 mm high (39.37 in.), and 100 mm (3.94 in.) thick. Similar distributions of reinforcement were also employed vertically and horizontally in the flange. The reinforcement was arranged to provide similar values of reinforcement ratios as those in Palermo and Vecchio's tests.⁹ Besides, additional confinements as suggested by Thomsen and Wallace¹¹ were provided at the edges of wall segments and junctures between web and flange to ensure the integrity of the wall. The material properties are shown in Table 1, which were selected based on the properties of squat walls in existing buildings. All specimens shared an identical geometry and reinforcement contents, while differences between them were axial loads and loading directions. The specimens were constructed with a stiff bottom slab with dimensions of 2100 x 2100 x 500 mm (82.68 x 82.68 x 19.69 in.) clamped to the laboratory strong floor. A heavily reinforced top slab with dimensions of 2700 x 2700 x 400 mm (106.30 x 106.30 x 15.75 in.) and a self-weight of 73.5 kN (16.5 kip) was also attached to the specimen by prestressing rods to transfer loads.

Test setup

All test specimens were subjected to a combination of quasi-static lateral displacements and axial loads to simulate an earthquake. Among them, HP5D0 and HP5D45 were imposed with a constant axial load of 700 kN (157.4 kip), which was considered representative of the loading condition of walls in low-rise buildings, while in HP0D0 and HP0D45, the extra axial loads were removed. As demonstrated in Fig. 2, axial loads were applied by a vertical hydraulic jack with one end connecting to the triangular steel frame. The load-

transfer system included three steel beams, a steel spreader plate, several steel rollers, four triangular steel plates, and 12 square steel plates, which were conservatively designed to ensure uniform axial load distribution on the wall cross section, and to accommodate horizontal movements of specimens. The lateral displacements were applied to the thick concrete slab by a horizontally mounted hydraulic jack. The loading procedure controlled by displacements was illustrated in Fig. 3, in which the lateral displacement increased by 1 mm (0.04 in.) every two cycles and terminated when the lateral strength of specimens dropped to less than 80% of the recorded maximum strength. Load cells were attached to jacks to measure the reaction forces. Besides, lateral braces consisting of several steel plates and load cells were mounted to the reaction walls to prevent out-of-plane deformations. All specimens shared the same loading apparatus.

Instrumentation

The test specimens were extensively installed with measuring devices both internally and externally. Strain gauges were used to capture strains on both longitudinal and horizontal reinforcement at critical locations. As seen in Fig. 4, the lateral displacement of the top slab was measured by linear variable differential transducers (LVDTs) mounted parallel to the horizontal jack. Flexural displacements were computed based on data captured by the vertical chains of LVDTs. Shear displacements were recorded by the LVDTs diagonally mounted, and calculated according to the method proposed by Hiraishi.¹² Sliding shear displacements were measured by a pair of LVDTs located near the bottom of the wall segment.

EXPERIMENTAL OBSERVATIONS AND RESULTS

Cracking pattern and failure mechanism

Figures 5 through 8 demonstrate the development of cracks on each surface of test specimens at three levels of drift ratios (DR), corresponding to the initial stage, the crack development stage, and the final failure stage, respectively. As seen from the figures, all specimens gained their initial cracks on four surfaces after the first run of the lateral loads, which was equal to a displacement of 1 mm (0.04 in.). After that, more diagonal cracks appeared, intersected with each other and gradually spread all over the web. HP5D0 with a higher axial load level demonstrated a steeper angle of inclined cracks on its web when compared with HP0D0. With regard to the cracking patterns on flanges, the majority of them were horizontal flexural cracks in specimens with orthogonal lateral loading, which gradually extended to the full length of the flange as the test progressed, while in HP0D45 and HP5D45 with skew lateral loading, shear

cracks preserved the predominant position. Besides, cracks only formed in the flange when it was in tension. As the test progressed to the final stage, diagonal cracks on the web became denser and spread to the entire surface, causing severe concrete spalling in specimens with orthogonal lateral loading. The specimens with skew lateral loading experienced reduced concrete spalling near the corner of the wall segment. The ultimate drift ratios at failure ranged from 0.9 to 1.5%.

HP0D0 failed at the DR of 1.1% as severe concrete crushing at the upper part of the wall web formed a major horizontal sliding plane, while the lateral strength still showed an ascending trend (Fig. 9). The first sign of the concrete crushing was witnessed as concrete spalling during the DR of 1.0%, and the crushing became visible at the DR of 1.1%. A similar premature failure was also observed in DP2 of Palermo and Vecchio⁹ as demonstrated in Fig. 10. These two specimens shared a similar section shape and were only imposed with the weight of the top slab. Unlike the typical sliding shear failure, which happened near the wall base, the sliding was observed in the upper part of the wall in both studies. Palermo and Vecchio⁹ suggested that the weaker concrete adopted in the web and the nature of thin wall section contributed to the major horizontal crack near the top slab. The explanation was reasonable because these two factors both contributed to the crushing of concrete at the top. However, the main reason behind this phenomenon was more related to the force-transfer mechanism in the test. In both experiments, a large and stiff top slab was used to transfer lateral loads to the wall segment. Ideally, horizontal displacements would be transferred assuming a perfect coordination existed between these two sections. Nonetheless, in the test, the existence of the stiff top slab and the incompatibility in transferring horizontal loads would cause a stress concentration in the upper part of the web. Under the cyclic loading, inclined cracks formed in each loading direction intersected with each other near the top, causing the concrete struts to become narrow. These struts were prone to crushing under such high stress. The concrete crushing further spread along the wall length, formed a weakened horizontal plane, and eventually led to the sliding of the wall.

Fig. 2—Test setup.

Fig. 3—Loading history. (Note: 1 mm = 0.04 in.)

Besides the horizontal sliding plane near the top slab, several vertical slip planes became visible on both sides of the web of HP5D0 at the DR of 0.9% as a result of the surface concrete spalling starting from the DR of 0.7%. A similar

Fig. 4—Arrangements of LVDTs in specimens.

failure mechanism was also recorded in DP1 of Palermo and Vecchio⁹ as presented in Fig. 11. They concluded that axial loads and stiff flanges restrained the opening of shear cracks. Thus, large shear stresses were introduced in the web by the significant shear distortion, leading to the formation of vertical slip planes. This explanation was considered plausible. As presented previously, the existence of additional axial loads increased the vertical vector of the internal stress in the web, thereby ascending the inclination of diagonal shear cracks. More importantly, the flexure-dominated flange restrained the shear-dominated web from moving, which meant a deformation incompatibility existed in the H-shaped squat walls. Under cyclic lateral loading, the web experienced tensile stresses in opposite directions to lateral displacements, which “tear” the web at the weak part, causing the formation of vertical slip planes. The model accounting for the deformation incompatibility was illustrated in the next chapter.

As for HP0D45 and HP5D45 with skew lateral loading, both specimens were able to sustain a ductile post-peak response before failure (Fig. 9). As lateral displacement further rose, a slight drop of shear strength was observed but in general the two specimens maintained their strengths for several cycles. Meanwhile, the diagonal shear cracks became much wider, especially in the upper corner of the

Fig. 5—Cracking patterns of HP0D0.

Fig. 6—Cracking patterns of HP0D45.

wall segment. As seen in the left of Fig. 12, inclined cracks resulted from negative loading direction concentrated at the left upper corner of the web, causing moderate concrete spalling starting from the DR of 0.7%, leading to the failure of HP0D45 at the DR of 0.9%. Reduced concrete spalling was witnessed in HP5D45 since the DR of 1.3%, where diagonal cracks in the junction of web and flange became evidently wider than those around the center of wall panel. With each successive cycle, the integrity between the web and the flange continued to diminish, which impaired the wall's ability to resist horizontal loads. Hence, the specimen gradually lost its lateral resistance by more than 30% of the maximum, and failed at the DR of 1.5%.

Hysteretic response

Hysteretic responses of test specimens are illustrated in Fig. 9, which were assessed by analyzing the backbone curves and the initial stiffness.

Backbone envelopes—The peaks of lateral loads at every displacement were used to construct the backbone envelopes, as shown in Fig. 13. It was evident that all specimens were able to sustain a ductile post-peak response prior to the failure except for HP0D0, in which a premature failure happened as the upper part of the web slid. It was observed that the lateral strength of HP5D45 with additional axial loads was larger

than its counterpart. A similar trend could also be traced in the group with orthogonal lateral loading, but their difference was less evident. This was because the lateral strength of HP5D0 was impaired by the vertical slip planes. With regard to the deformation capacity, HP5D45 outperformed HP0D45 with the number of 1.5% compared with 0.8%. However, the specimens with orthogonal lateral loading demonstrated nearly the same capability of deformation.

Effective stiffness—The effective stiffness was determined based on a point from the force-displacement envelope, when either the first yielding occurred in the longitudinal reinforcement or the maximum compressive strain of concrete reached 0.002. Reduction factors of gross moment of inertia are usually employed by code standards to account for the loss of flexural and shear rigidities. ACI 318-14¹⁰ suggests reduction factors of 0.35 and 0.70 for cracked and uncracked concrete, respectively. In ASCE 43-05,¹³ no reduction is recommended for uncracked concrete and 0.5 is recommended for cracked concrete. Figure 13 demonstrates the effective stiffness of test specimens, which yielded the value of $E_c I_e / E_c I_g$ as 0.010, 0.020, 0.012, and 0.022 for HP0D0, HP0D45, HP5D0, and HP5D45, respectively. Overestimations were found by using the aforementioned code provisions. This could be due to the drastic increase of moment of inertia of gross section brought by flange segments.

Fig. 7—Cracking patterns of HP5D0.

Because an evident shear lag effect was found in the specimens (shown in the next section), it was inappropriate to adopt the moment of inertia of the whole uncracked section in estimating the effective stiffness of H-shaped walls. It was also noticed that axial loads brought growth on the effective stiffness, which was consistent with the observations from other researchers.^{14–18} The effective stiffness increased by 16% and 11% in two groups as extra axial loads were imposed. As for the influence of loading direction, a growth of effective stiffness more than 20% was also recorded when the direction of lateral loads changed from parallel to web to 45 degrees from the web.

Deformation component

Top deformations of structural walls mainly comprised three components: flexural deformation, shear deformation, and sliding deformation. Figure 14 illustrates the ratios of three components to the total top displacement at different drift ratios, in which an unaccounted portion was also reported because the summation of the other three could not always reach a perfect 100%.

The predominant position of shear deformation was apparent because it accounted for the largest portion of total deformation during the most time of the test. A maximum value of 62% was witnessed in HP5D0 in both loading direc-

tions. It was also evident that the portion of shear deformation increased in all tests, whereas declining trends were observed in the flexural deformation. For specimens with orthogonal lateral loading, the shear part was constantly and evidently larger than the flexural part throughout the whole experiment, while for specimens with skew lateral loading, the contribution of top deformation from shear was initially smaller than that of flexure, but ended up equal to or higher than its counterpart. This was because the principal shear force was transferred by multiple segments in the former, while only one segment mainly functioned in the latter. As for the influence of axial loads, a slight growth of the shear deformation was observed between HP0D0 and HP5D0, whereas no evident impact was found in specimens with skew lateral loading. The unaccounted part remained at a low value during the whole tests.

As for the sliding shear component, a continuous rising trend was witnessed in all specimens. Starting from 3 to 5%, the number increased to 8 to 11% at the end of the test. Compared with specimens with additional axial loads, those only subjected to the self-weight of the top slab experienced a more rapid growth of the sliding shear deformation, which was in accordance with previous findings on squat walls.¹⁹ This was because axial loads could effectively limit the width of cracks, thereby slowing the development of horizontal

Fig. 8—Cracking patterns of HP5D45.

Fig. 9—Lateral load-displacement curves of specimens. (Note: 1 mm = 0.04 in.)

cracking planes. As for the impact of the loading direction, no significant variation was found between the orthogonally loaded group and the skew-loaded group, suggesting that the

change of the loading direction did not play a favorable role in controlling of the sliding shear.

Fig. 10—HP0D0 and DP2 at failure.

Fig. 11—HP5D0 and DP1 at failure.

Fig. 12—HP0D45 and HP5D45 at failure.

Fig. 13—Backbone curves and effective stiffness of specimens. (Note: 1 mm = 0.04 in.)

Strain profile

Figure 15 presents the strain variations along the bottom of the longitudinal reinforcement in the flange. For those with orthogonal lateral loading, data of the left flange were collected; for those with skew lateral loading, data of the right flange were used. As seen from the figure, the longitu-

dinal reinforcing bar located in the juncture of the web and flange yielded in specimens with orthogonal lateral loading. Meanwhile, in the group with skew lateral loading, the outmost longitudinal reinforcing bar yielded at the end of the test. A significant shear lag effect was detected in HP0D0 and HP5D0. For instance, the strain of the reinforcing bar in

Fig. 14—Displacement components of specimens.

Fig. 15—Strain profiles of longitudinal reinforcement along flange of specimens. (Note: 1 m = 39.37 in.)

the junction of the web and flange was as much as 320% of that of the longitudinal reinforcement located in the flange tip at the final test stage in HP0D0. As for HP5D0, the shear lag effect was more evident because strains of outmost longitudinal reinforcement were almost zero when the flange was in tension, indicating the outer part of the flange was hardly involved in resisting lateral loads. Similar distributions of strains of longitudinal reinforcement were also reported in other experiments on walls with larger flange.^{11,14,20,21} With regard to specimens under skew lateral loading, the shear lag effect was less obvious but could still be traced as the tensile strains of longitudinal reinforcement showed an increasing trend toward the flange tip. It should also be noticed that the maximum strain was observed at 240 mm (9.45 in.) from the flange tip, which was in accordance with the results of section analyses.¹⁴

FAILURE MECHANISM AND SHEAR STRENGTH

Vertical slip plane

As mentioned in the observation of experimental results, several vertical slip planes gradually formed from the drift ratio of 0.7% in the web of HP5D0. An identical phenomenon was also detected in DP1 of Palermo and Vecchio.⁹ These two specimens shared a similar section shape and reinforcement contents, and were both subjected to axial loads slightly above $0.05A_gf'_c$. The flange of both specimens remained relatively intact throughout the test. The majority of cracks on their flanges were flexural cracks, which extended to the full length of the flange. Thus, it could be concluded the flexure dominated the responses of flanges. The web of these two specimens, on the other hand, was mainly controlled by shear. It was revealed not only in the cracking pattern as presented in Fig. 7 but also in the illustration of

Fig. 16—Deformation incompatibility in HP5D0.

deformation components, where the shear contributed more than 60% of total deformation in both loading directions.

Figure 16 demonstrates the deformation incompatibility during the test in HP5D0. As seen in the left, ideally both flange and web segments experienced the same deformation at all heights. However, in reality, the flange was mainly subjected to the flexural distortion, while the shear distortion was concentrated in the web, which would inevitably cause an incompatibility in deformation between these two segments as marked in black. The deformation of flange along the wall height is written as

$$\omega_f = \frac{Vx^2}{6E_c I_g} (3h_w - x) \quad (1)$$

where V is the lateral reaction force; x is the distance from the base to the top, ranging from 0 to h_w ; E_c is the elastic modulus of concrete; I_g is the moment of inertia of flange section; and h_w is the wall height. Because the flange and web had the same deformation at the top, assuming the web was only subjected to the shear distortion, the deformation of web along the wall height is expressed as

$$\omega_w = \frac{Vh_w^2 x}{3E_c I_g} \quad (2)$$

Hence, the deformation incompatibility is obtained by subtracting Eq. (1) from Eq. (2) as

$$\Delta_\omega = \omega_w - \omega_f = \frac{V}{6E_c I_g} (x^3 - 3h_w x^2 + 2h_w^2 x) \quad (3)$$

To understand the variation of deformation incompatibility along the height, it was necessary to solve the derivative of the aforementioned function.

$$\Delta_\omega = \omega_w - \omega_f = \frac{V}{6E_c I_g} (x^3 - 3h_w x^2 + 2h_w^2 x) \quad (4)$$

$$x = \left(1 \pm \frac{\sqrt{3}}{3}\right) h_w = 0.42h_w \text{ or } 1.58h_w \quad (5)$$

Because the predefined range of x was from 0 to h_w , it could be concluded that the deformation incompatibility gradually increased from the wall base as the height rose, reaching its maximum at the height of $0.42h_w$, and declined as the height further ascended. Accordingly, the horizontal tensile force created by the deformation incompatibility also gained its

maximum around the middle height, which was in accordance with the position of vertical slip planes in both tests. Moreover, the imposed axial loads caused inclined shear cracks to become steeper. Under cyclic loading, more intersected cracks accumulated in the center of the web panel, as shown in Fig. 16. Therefore, a weakened vertical plane would form in this particular area with concrete spalling as observed in the test.

Shear strength

The cracking pattern not only reveals seismic responses but also is crucial in studying the force-transfer mechanism in RC structures. Based on the idealized cracking pattern and the extracted free body diagram, Moehle²² proposed a series of expressions for the prediction of peak shear strength of low-rise RC walls. Luna et al.²³ later examined and improved this methodology using experimental data of 12 squat walls. In this study, the free body diagram was also employed to develop equations for the peak shear strength of H-shaped squat RC walls with orthogonal lateral loading.

The cracking pattern and failure mechanisms have been thoroughly addressed in the previous section. As seen from Fig. 17, three segments could be divided from the cracking pattern of HP0D0 and HP5D0. All these segments were imposed with axial stresses and shear stresses. Cracks were assumed to spread at a constant angle θ with respect to the horizontal. h_w is the height of wall, t_f is the thickness of flange, and l_w is the length of the web.

The free body diagram of HP0D0 is illustrated in Fig. 18. As seen in the segment A, l_f is the length of the flange, t_w is the thickness of the web, v is the uniform shear stress transferring at the top, p is the uniform axial stress acting on the top, F_{vi} and F_{hi} represent the force carried by individual longitudinal and horizontal reinforcement, τ_s is the friction stress along the diagonal crack, and F_{cx} and F_{cy} are the horizontal and vertical components of reaction force of the compression zone. The total force carried by the longitudinal web reinforcement along the diagonal crack could be represented as

$$F_v = k_1 \rho_v t_w f_{yv} \frac{h_w}{\tan \theta} \quad (6)$$

where k_1 is a coefficient accounting for the stress in the longitudinal reinforcement in the web; ρ_v is the longitudinal reinforcement ratio in the web; and f_{yv} is the yield stress of longitudinal reinforcement in the web. Similarly, the total force carried by the horizontal reinforcement is represented by F_h . Summing moments about Point A in the lower right corner

Fig. 17—Different sections and idealized cracking pattern of HP0D0 and HP5D0.

Fig. 18—Free body diagram of HP0D0.

$$vt_w \left(\frac{h_w}{\tan \theta} + t_f \right) h_w = \frac{1}{2} F_{\text{G}} t_f + \frac{1}{2} F_h h_w + \frac{1}{2} F_v \frac{h_w}{\tan \theta} + \frac{1}{2} p t_w \frac{h_w^2}{\tan^2 \theta} - \frac{1}{2} p l_f t_f^2 \quad (7)$$

Hence, the shear strength carried by Segment A is obtained as

$$V_a = \frac{1}{2}F_{cy}\frac{t_f}{h_w} + \frac{1}{2}F_h + \frac{1}{2}F_v\frac{1}{\tan\theta} + \frac{1}{2}pt_w\frac{h_w}{\tan^2\theta} - \frac{1}{2}pt_f^2\frac{l_f}{h_w} \quad (8)$$

Segment B was further divided into several parts with a width of x , where F_c is the reaction force. Summing moments about the mid-bottom of the segment gives

$$v t_w x h_w = k_1 \rho_v x t_w f_{yv} \frac{h_w}{\tan \theta} + p x t_w \frac{h_w}{\tan \theta} + F_s x \sin \theta \quad (9)$$

where F_s is the force associated with the friction. Hence, the shear strength carried by Segment B is obtained as

$$V_b = k_l \rho_v t_w f_{yv} \left(l_w - 2t_f - \frac{h_w}{\tan \theta} \right) \frac{1}{\tan \theta} + p t_w \left(l_w - 2t_f - \frac{h_w}{\tan \theta} \right) \frac{1}{\tan \theta} + F_s \left(l_w - 2t_f - \frac{h_w}{\tan \theta} \right) \frac{\sin \theta}{h_w} \quad (10)$$

As for Segment C, because the shear lag effect existed, not all longitudinal reinforcing bar in the flange reached yielding at the peak shear strength. A coefficient k_2 ranging from 0 to 1 is designated for the variation of stress in these bars

$$F_t = k_2 \rho_f t_f l_f f_{vf} \quad (11)$$

where f_{yf} is the yield stress of longitudinal reinforcement in the flange; and ρ_f is longitudinal reinforcement ratio in the flange. Summing moments about Point C in the lower left corner of Segment C yields

$$vt_w t_f h_w = \left(pl_f t_f + F_t \right) \left(\frac{h_w}{\tan \theta} + \frac{1}{2} t_f \right) - \frac{1}{2} F_h h_w - \frac{1}{2} F_v \frac{h_w}{\tan \theta} \quad (12)$$

where k_2 is a coefficient accounting for the stress in the longitudinal reinforcement in the flange; and ρ_f is the longitudinal reinforcement ratio in the flange. Therefore, the shear strength carried by Segment C is obtained as

$$V_c = (pl_f t_f + F_t) \left(\frac{1}{\tan \theta} + \frac{1}{2} \frac{t_f}{h_w} \right) - \frac{1}{2} F_h - \frac{1}{2} F_v \frac{1}{\tan \theta} \quad (13)$$

The total lateral resistance of HP0D0 is the summation of contributions from the aforementioned three segments

Fig. 19—Free body diagram of HP5D0.

$$V_{cal} = V_a + V_b + V_c = \frac{1}{2} F_{cy} \frac{t_f}{h_w} + F_t \left(\frac{1}{\tan \theta} + \frac{1}{2} \frac{t_f}{h_w} \right) + (F_v + F_s \sin \theta) \left(l_w - 2t_f - \frac{h_w}{\tan \theta} \right) \frac{1}{h_w} + p \left(l_f t_f + t_w l_w - 2t_w t_f - \frac{1}{2} \frac{t_w h_w}{\tan \theta} \right) \frac{1}{\tan \theta} \quad (14)$$

In this equation, the vertical component of the reaction force in Segment A is

$$F_{cy} = t_f l_f f'_c \quad (15)$$

As for the friction, F_s was mainly taken by the aggregate interlock and the dowel action across the cracks. In the models proposed by Moehle²² and Luna et al.,²³ the normal and shear stresses along diagonal cracks were conservatively assumed to be resisted only by the reinforcement. Also, Gulec and Whittaker²⁴ found considerable uncertainties existed in the evaluation of the magnitude of the friction. The same assumption was adopted herein and the friction force was set equal to zero. Thus, the total lateral resistance (peak shear strength) of HP0D0 can be further expressed as

$$V_{cal} = \frac{1}{2} f'_c \frac{t_f l_f}{h_w} + k_1 \rho_v t_w l_f f_y \left(l_w - 2t_f - \frac{h_w}{\tan \theta} \right) \frac{1}{\tan \theta} + k_2 \rho_f t_f l_f f_{yf} \left(\frac{1}{\tan \theta} + \frac{1}{2} \frac{t_f}{h_w} \right) + t_w l_w - 2t_w t_f - \frac{1}{2} \frac{t_w h_w}{\tan \theta} \frac{1}{\tan \theta} \quad (16)$$

As for HP5D0, the cracking pattern was quite different. Figure 19 shows the free body diagram of HP5D0, where the vertical slip plane was represented by a vertical line, and shear cracks were shown as two diagonal lines. From the observation in HP5D0 as well as DP1, the vertical slip plane was assumed to locate in the middle of the web panel taking over one-third of the wall height. As seen in the figure, several notations shared identical meanings as defined previously. F_{s1} is the friction force along the inclined crack, F_{s2} is the friction force along the vertical slip plane, l_1 is the horizontal distance between the vertical slip plane, the flange τ_{s1}

is the friction stress along the shear crack, and τ_{s2} is the friction stress along the vertical slip plane. Similar approaches were continually used by summing moments at Point A, the mid-bottom of Segment B, and Point C, respectively. The total lateral resistance was obtained as the summation of the contributions from three segments as

$$V_{cal} = \frac{1}{2} f'_c \frac{t_f^2 l_f}{h_w} + k_1 \rho_v t_w l_f f_{yv} (2l_w - 4t_f - 4l_1) \frac{1}{h_w} + k_2 \rho_f t_f l_f f_{yf} \left(2 \frac{l_1}{h_w} + \frac{1}{2} \frac{t_f}{h_w} \right) + 2p \frac{l_1}{h_w} \left(l_w - 2t_f - l_1 + l_f \frac{t_f}{t_w} \right) \quad (17)$$

Comparison of analytical and experimental results

The equations derived from the free body diagram were verified with experimental data. The cracking pattern of HP0D0 and DP2 represented the typical cracking pattern of squat RC walls. Thus, 30 H-shaped RC squat walls with a similar cracking pattern were also selected for the comparison, as presented in Table 2, which comprised of the research work of Barda et al.,³ Paulay et al.,² Sato et al.,⁴ and Saito et al.⁵ Equations in Section 18.10 of ACI 318-14¹⁰ for the seismic design of structural walls were also incorporated in the table. The angle of inclined crack θ , in Eq. (16), was measured directly from the cracking pattern. The two reduction factors k_1 and k_2 accounting for the stress in the longitudinal reinforcement were obtained based on the strain profiles, which were 0.4 and 0.7 for HP0D0. For those experiments that did not provide the strain data, k_2 was calculated from the effective flange width defined by ACI 318-14, which assumed the longitudinal reinforcing bar located within a distance of one-quarter of wall height was yielded. All longitudinal bars in the web were assumed yielded, which indicated k_1 equal to 1. As seen from the table, compared with the ACI equations (V_{ACI}), the proposed equation (V_{cal}) could more accurately predict the peak shear strength of H-shaped RC squat walls, which was revealed in the smaller value of coefficient of variation and mean value closer to 1.

Table 2—Comparisons of measured and calculated peak shear strength

Researcher	Specimen	h_w , mm	l_w , mm	t_w , mm	l_f , mm	t_f , mm	θ , deg	f'_c , MPa	f_{yv} , MPa	f_{yf} , MPa	ρ_v , %	ρ_f , %	P , kN	V_{ACI}/V_{test}	V_{cal}/V_{test}
Ma and Li ²¹	HP0D0	1000	1500	100	1500	100	50	35.9	554	554	0.97	1.26	74	0.68	1.21
Barda et al. ³	B3-2	953	1905	102	610	102	32	27.0	545	414	0.50	4.10	0	0.67	1.07
	B4-3	953	1905	102	610	102	31	23.5	535	527	0.50	4.10	0	0.23	1.47
	B5-4	953	1905	102	610	102	36	19.0	0	527	0.00	4.10	0	0.99	1.73
	B6-4	953	1905	102	610	102	38	21.2	496	529	0.25	4.10	0	0.80	1.36
	B7-5	476	1905	102	610	102	34	25.7	531	539	0.50	4.10	0	0.64	1.51
Paulay et al. ²	Wall 3	1700	3000	100	500	100	43	26.0	315	300	0.37	1.81	0	1.62	0.58
Saito et al. ⁵	W12-1	1000	2120	120	500	120	45	35.0	369	369	1.32	4.72	706	0.47	0.83
	W12-2	1000	2120	120	500	120	45	38.0	369	369	0.90	4.72	698	0.49	0.81
	W12-3	1000	2120	120	500	120	45	36.0	369	369	1.69	4.72	713	0.50	0.94
	W12-4	1000	2120	120	500	120	45	36.0	369	369	1.32	4.72	350	0.51	0.80
	W12-5	1000	2120	120	500	120	45	40.0	369	369	1.32	4.72	706	0.50	0.83
	W15-1	1000	2120	150	500	120	45	25.0	369	369	1.06	4.72	830	0.60	1.04
Sato et al. ⁴	24M 8-30	1600	2150	150	1000	150	48	38.0	296	296	0.80	0.80	1155	0.75	0.78
	24M 8-40	1600	2150	150	1000	150	48	35.8	422	422	0.60	0.60	1155	0.75	0.75
	24M 8-50	1600	2150	150	1000	150	48	35.0	528	528	0.48	0.48	1155	0.74	0.75
	24M 6-30	1200	2150	150	1000	150	48	40.1	296	296	0.80	0.80	1155	0.61	0.72
	24M 6-40	1200	2150	150	1000	150	48	40.0	422	422	0.60	0.60	1155	0.60	0.70
	36L 8-30	1600	2150	150	1000	150	48	24.5	296	296	1.20	1.16	1155	0.74	0.78
	36L 8-40	1600	2150	150	1000	150	48	27.8	422	422	0.90	0.90	1155	0.77	0.82
	36M 8-30	1600	2150	150	1000	150	48	39.3	296	296	1.20	1.16	1155	0.87	0.80
	36M 8-40	1600	2150	150	1000	150	48	38.8	422	422	0.90	0.90	1155	0.82	0.77
	36M 8-50	1600	2150	150	1000	150	48	37.5	528	528	0.72	0.72	1155	0.82	0.79
	36M 6-30	1200	2150	150	1000	150	48	33.4	296	296	1.20	1.16	1155	0.69	0.75
	36M 6-40	1200	2150	150	1000	150	48	34.6	422	422	0.90	0.90	1155	0.66	0.74
	48M 8-30	1600	2150	150	1000	150	48	27.4	296	296	1.60	1.60	1155	0.70	0.84
	48M 8-40	1600	2150	150	1000	150	48	27.5	422	422	1.20	1.16	1155	0.69	0.83
	48M 8-50	1600	2150	150	1000	150	48	28.0	528	528	0.96	0.96	1155	0.70	0.85
	48H 8-30	1600	2150	150	1000	150	48	41.8	296	296	1.60	1.60	1155	0.76	0.78
	48H 8-40	1600	2150	150	1000	150	48	43.1	422	422	1.20	1.16	1155	0.75	0.78
	48H 8-50	1600	2150	150	1000	150	48	44.6	528	528	0.96	0.96	1155	0.74	0.76
Palermo and Vecchio ⁹	DP2	2340	3075	75	3045	100	45	18.8	605	605	0.82	0.35	260	0.92	0.92
													Average	0.712	0.908
													COV	0.311	0.292

Notes: 1 mm = 0.04 in.; 1 MPa = 145 psi; 1 kN = 0.225 kip.

As for Eq. (17), only two H-shaped specimens in the literature (HP5D0 and DP1) were found to match the specific cracking pattern. Although the calculation yielded a mean value of 1.016, the data incorporated was considered too sparse. Thus, further tests on H-shaped squat walls with axial loads and cyclic lateral loads are needed to examine the proposed equation.

SUMMARY AND CONCLUSIONS

An experimental program was carried out on four H-shaped RC squat walls under the simulated gravity and cyclic lateral loads at NTU. Seismic responses of specimens

were presented and discussed in detail. A reasonable analytical model was subsequently proposed to account for the vertical slip planes resulting from the deformation incompatibility between web and flange. Based on the cracking pattern, free body diagrams were also employed to provide equations for the peak shear strength of H-shaped squat walls. The key findings of this study are:

1. The intersection of diagonal cracks caused severe concrete spalling at the upper part of the web, formed a major horizontal sliding plane, and eventually led to the failure of HP0D0. Several vertical slip planes as found in DP1 of Palermo and Vecchio⁹ also occurred in HP5D0, which could

be attributed to the combined effect of axial loads and stiff flange and the consequent deformation incompatibility. The other two specimens with skew lateral loading were able to sustain a ductile post-peak response for few cycles, and failed as the lateral strength dropped more than 30% of the maximum with moderate concrete spalling around the corner of wall segments.

2. Due to improper considerations about the flange of H-shaped squat walls, code provisions overestimated the effective stiffness of specimens. Additional axial loads increased the effective stiffness by approximately 14% in two groups. Also, as corroborated by the slender non-rectangular walls,¹⁴ specimens with skew lateral loading had bigger values of effective stiffness than specimens with orthogonal lateral loading.

3. The shear deformation that dominated the response of squat walls was found more evident in the specimens with orthogonal lateral loading, with a maximum value of 62% recorded in HP5D0. Continuous growths of both shear displacement and sliding shear displacement were observed in all specimens, while the flexural deformation decreased during the same period, which was in accordance with previous findings on squat walls.¹⁹ Moreover, the axial load was observed effective in controlling the development of the shear-related deformation.

4. The web of HP5D0 and DP1 was dominated by shear, while their flanges were controlled by flexure. Thus, deformation incompatibilities existed between the web and flange when specimens were imposed with lateral displacements. Further analytical investigations revealed that the deformation incompatibility reached its maximum at the height of $0.42h_w$, leading to the vertical slip planes at the same altitude along the weakened area.

5. The free body diagrams derived from the cracking pattern of HP0D0 and HP5D0 were used to account for the force-transfer mechanism and to deduce equations for the peak shear strength of H-shaped squat walls. Compared with ACI 318-14, the proposed equation could better predict the peak shear strength of H-shaped squat walls with a similar cracking pattern as HP0D0. As for the equation developed based on HP5D0, the data available in the literature was insufficient for the validation. Thus, more experimental data is needed for H-shaped RC squat walls under axial loads and lateral cyclic loads.

AUTHOR BIOS

Jiaying Ma is an Assistant Professor in the school of Civil Engineering and Architecture at Ningbo Institute of Technology, Zhejiang University, Ningbo, China. He received his PhD from Nanyang Technological University, Singapore. His research interests include earthquake engineering and reinforced concrete squat walls.

ACI member **Bing Li** is an Associate Professor in the School of Civil and Environmental Engineering at Nanyang Technological University. He is a member of ACI Committee 377, Performance-Based Structural Integrity & Resilience of Concrete Structures, and Joint ACI-ASCE Committees 352, Joints and Connections in Monolithic Concrete Structures, and 441, Reinforced Concrete Columns.

REFERENCES

1. Syed, S., and Gupta, A., "Seismic Fragility of RC Shear Walls in Nuclear Power Plant Part 1: Characterization of Uncertainty in Concrete Constitutive Model," *Nuclear Engineering and Design*, V. 295, 2015, pp. 576-586. doi: 10.1016/j.nucengdes.2015.09.037

2. Paulay, T.; Priestley, M. J. N.; and Syng, A. J., "Ductility in Earthquake Resisting Squat Shearwalls," *ACI Journal Proceedings*, V. 79, No. 4, July-Aug. 1982, pp. 257-269.

3. Barda, F.; Hanson, J. M.; and Corley, W. G., "Shear Strength of Low-Rise Walls with Boundary Elements," *Reinforced Concrete Structures in Seismic Zones*, SP-53, N. M. Hawkins and D. Mitchell, eds., American Concrete Institute, Farmington Hills, MI, 1977, pp. 149-202.

4. Sato, S.; Ogata, Y.; Yoshizaki, S.; Kanata, K.; Yamaguchi, T.; Nakayama, T.; Inada, Y.; and Kadoriku, J., "Behavior of Shear Wall Using Various Yield Strength of Rebar, Part 1: An Experimental Study," *Tenth International Conference on Structural Mechanics in Reactor Technology*, Anaheim, CA, 1989.

5. Saito, H.; Kikuchi, R.; Kanechika, M.; and Okamoto, K., "Experimental Study on the Effect of Concrete Strength on Shear Wall Behavior," *Tenth International Conference on Structural Mechanics in Reactor Technology*, Anaheim, CA, 1989.

6. Naze, P. A., and Sidaner, J. F., "Presentation and Interpretation of SAFE Tests: Reinforced Concrete Walls Subjected to Shearing," *Sixteenth International Conference on Structural Mechanics in Reactor Technology*, Washington, DC, 2001.

7. Mo, Y. L., and Chan, J., "Behavior of Reinforced-Concrete-Framed Shear Walls," *Nuclear Engineering and Design*, V. 166, No. 1, 1996, pp. 55-68. doi: 10.1016/0029-5493(96)01244-7

8. Kitada, Y.; Akino, K.; Terada, K.; Aoyama, H.; and Miller, A., "Report on Seismic Shear Wall International Standard Problem Organized by OECD/NEA/CSNI," *Fourteenth International Conference on Structural Mechanics in Reactor Technology*, Lyon, France, 1997.

9. Palermo, D., and Vecchio, J. J., "Behavior of Three-Dimensional Reinforced Concrete Shear Walls," *ACI Structural Journal*, V. 99, No. 1, Jan.-Feb. 2002, pp. 81-89.

10. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)," American Concrete Institute, Farmington Hills, MI, 2014, 519 pp.

11. Thomsen, J. H. IV, and Wallace, J. W., "Displacement-Based Design of Slender Reinforced Concrete Structural Walls-Experimental Verification," *Journal of Structural Engineering*, ASCE, V. 130, No. 4, 2004, pp. 618-630. doi: 10.1061/(ASCE)0733-9445(2004)130:4(618)

12. Hiraishi, H., "Evaluation of Shear and Flexural Deformations of Flexural Type Shear Walls," *Bulletin of the New Zealand National Society for Earthquake Engineering*, V. 17, No. 2, 1984, pp. 135-144.

13. ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," American Society of Civil Engineers, Reston, VA, 2005, 96 pp.

14. Zhang, Z., and Li, B., "Seismic Performance Assessment of Slender T-Shaped Reinforced Concrete Walls," *Journal of Earthquake Engineering*, V. 20, No. 8, 2016, pp. 1342-1369. doi: 10.1080/13632469.2016.1140097

15. Pham, T. P., and Li, B., "Seismic Performance Assessment of L-Shaped Reinforced Concrete Columns," *ACI Structural Journal*, V. 112, No. 6, Nov.-Dec. 2015, pp. 667-678. doi: 10.14359/51688053

16. Li, B., and Xiang, W., "Effective Stiffness of Squat Structural Walls," *Journal of Structural Engineering*, ASCE, V. 137, No. 12, 2011, pp. 1470-1479. doi: 10.1061/(ASCE)ST.1943-541X.0000386

17. Li, B., and Lim, C. L., "Tests on Seismically Damaged Reinforced Concrete Structural Walls Repaired Using Fiber-Reinforced Polymers," *Journal of Composites for Construction*, ASCE, V. 14, No. 5, 2010, pp. 597-608. doi: 10.1061/(ASCE)CC.1943-5614.0000110

18. Sengupta, P., and Li, B., "Seismic Fragility Evaluation of Lightly Reinforced Concrete Walls," *Journal of Earthquake Engineering*, V. 20, No. 5, 2010, pp. 809-840. doi: 10.1080/13632469.2015.1104755

19. Li, B.; Pan, Z.; and Xiang, W., "Experimental Evaluation of Seismic Performance of Squat RC Structural Walls with Limited Ductility Reinforcing Details," *Journal of Earthquake Engineering*, V. 19, No. 2, 2015, pp. 313-331. doi: 10.1080/13632469.2014.962669

20. Brueggen, B.; French, C.; Jung, N.; and Nakaki, S., "Non-Rectangular Reinforced Concrete Shear Walls: Design Issues and Performance," *8th National Conference on Earthquake Engineering*, San Francisco, CA, 2006.

21. Ma, J., and Li, B., "Seismic Behavior of L-shaped RC Squat Walls under Various Lateral Loading Directions," *Journal of Earthquake Engineering*, 2017, pp. 1-22. doi: 10.1080/13632469.2017.1326424

22. Moehle, J., *Seismic Design of Reinforced Concrete Buildings*, McGraw-Hill Education, New York, 2015, 760 pp.

23. Luna, B. N.; Rivera, J. P.; and Whittaker, A. S., "Seismic Behavior of Low-Aspect-Ratio Reinforced Concrete Shear Walls," *ACI Structural Journal*, V. 112, No. 5, Sept.-Oct. 2015, pp. 593-603. doi: 10.14359/51687709

24. Gulec, C. K., and Whittaker, A. S., "Empirical Equations for Peak Shear Strength of Low Aspect Ratio Reinforced Concrete Walls," *ACI Structural Journal*, V. 108, No. 1, Jan.-Feb. 2011, pp. 80-89.