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<th>Experimental assessment of t-shaped reinforced concrete squat walls</th>
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<td>Author(s)</td>
<td>Ma, Jiaxing; Zhang, Zhongwen; Li, Bing</td>
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Experimental Assessment of T-Shaped Reinforced Concrete Squat Walls

by Jiaxing Ma, Zhongwen Zhang, and Bing Li

Reinforced concrete (RC) T-shaped walls have been studied by many researchers over the past decades due to their popularity. Among them, however, few investigations are conducted regarding T-shaped squat walls, especially for their seismic behaviors under nonprincipal bending action. To build the database and improve the understanding of structural walls, reversed cyclic tests of four RC T-shaped squat walls were conducted under displacement control. The variables were axial loads and lateral loading directions. Seismic responses of specimens were presented and assessed in detail from various aspects. Nonlinear section analyses and finite element modeling were also performed to facilitate investigations. The results indicated a significant shear lag effect exited in some T-shaped squat walls, which distinctly affected the strength and stiffness of test specimens. It was also found the impact of the shear lag effect increased with additional axial loads, and decreased as the test progressed.

Keywords: effective stiffness; lateral loading direction; reinforced concrete; seismic performance; shear lag effect; squat walls; T-shaped.

INTRODUCTION

Reinforced concrete (RC) T-shaped structural walls are the primary lateral load-carrying element in many structures designed for protective purposes.1 These walls are favored by structural engineers due to their significant stiffness in both principal directions, which contributes to the resistance to biaxial bending action during strong earthquake actions. Compared with rectangular RC walls, the interaction between the web and flange complicates their seismic behaviors. Scholars2-12 have done many research investigations in this area. Thomsen and Wallace5 carried out an experimental program containing two one-fourth-scale T-shaped slender walls, concluding that additional considerations such as special boundary element transverse reinforcement should be given to T-shaped walls to ensure the prevention of brittle failure. Brueggen et al.13 later conducted tests on two large-scale T-shaped walls with multi-directional loading, concluding that the shear lag effect could significantly impact flanged walls and required further studies. Zhang and Li14 also tested six T-shaped slender walls with a shear span ratio of 2.5. Among them, three were imposed with skew horizontal loading 45 degrees from the web and the other half was symmetrically loaded. They pointed out the existence of lateral loads in one principal direction generally reduced the lateral resistance of walls in the principal loading direction. Most of these studies, nonetheless, mainly focused on slender walls. As for the T-shaped squat walls that are extensively employed in low-rise buildings as well as nuclear facilities, the experimental data are rather sparse.

Furthermore, current seismic design procedures are developed for rectangular slender walls and have been relatively invariable for many years.14 Wood15 and Gulec et al.16 concluded in their studies that procedures available in code provisions resulted in a substantial scatter in predicting the peak shear strength of nonrectangular squat walls. Zhang and Li14 and Hassan and El-Tawil17 found that the effective flange width defined by standards does not correlate well with the experimental and numeric results of nonrectangular walls. Thus, further exploration and refinement on current codes are needed.

As an effort to better understand the seismic behaviors of nonrectangular RC squat walls, especially T-shaped walls, an experimental program containing four T-shaped wall specimens was conducted at Nanyang Technological University (NTU), Singapore. This paper includes both experimental and analytical investigations. First, a test program is introduced and then observations of the experiments are described in detail. Analyses are then carried out through various approaches.

RESEARCH SIGNIFICANCE

Compared with T-shaped slender walls, few studies have been carried out regarding the equally important T-shaped squat walls, especially for their seismic behaviors under nonprincipal bending action. This paper presents experimental data regarding four T-shaped squat walls tested at NTU. A significant shear lag effect was observed in T-shaped squat walls, which increased with more axial loads, and decreased as the drift ratio rose. Investigations also revealed the effective stiffness of specimens was overestimated by building codes and methods available in the literature.

TEST PROGRAM

Details of specimens

This experimental program included a total of four specimens named TP0D0, TP0D45, TP5D0, and TP5D45. All specimens had the same dimensions, as illustrated in Fig. 1. Based on the provisions of ACI 318-1418 and the lab setup, specimens were designed to represent squat walls in nuclear facilities. Limited to the dimensions of the testing apparatus, dimensions of the tested specimens are signifi-
cantly reduced compared to walls in real buildings. The sizes
of the reinforcing bars and aggregates are not to scale. These
modifications could affect the shear strength and failure
mechanism of the specimens. Two batches of Grade 40
concrete with measured compressive strengths of 35.0 and
36.2 MPa (5.1 and 5.3 ksi) were used. Reinforcing bars with
a diameter of 10 mm (0.39 in.) were employed as longitudi-
nal reinforcement in both web and flange with the relevant
reinforcement ratios of 1.12% and 1.26%, respectively. The
horizontal reinforcement was provided by reinforcing bars
with a diameter of 6 mm (0.24 in.) at a spacing of 80 mm
(3.15 in.), which yielded a reinforcement ratio of 0.71%. The
properties of materials are summarized in Table 1. Additional
confinements were provided at the edges of wall segments
and junctures between web and flange to ensure the integrity
of the walls. A heavily reinforced concrete slab with dimen-
sions 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 attached to spec-
imens by prestressing rods to transfer loads. The differences
between each specimen were axial loads and lateral loading
directions, which are presented in Table 1. For TP0D0 and
TP0D45, the axial load was solely the self-weight of the
top slab; for TP5D0 and TP5D45, an addition axial load of
500 kN (112.4 kip) was imposed.

**Test setup and instrumentation**

All test specimens were subjected to quasi-static horizontal
cyclic load reversals to simulate earthquakes. As mentioned
previously, two specimens were imposed with a constant axial
load of approximately 0.05Ago' by a hydraulic jack. The load
transfer system consisted of three steel beams, a steel spreader
plate, several steel rollers, four triangular steel plates, and 12
square steel plates, which are shown in Fig. 2. This setup was
designed to ensure a uniform axial load distribution on the wall
cross section and to accommodate the horizontal movement
of wall specimens due to lateral displacements. The revers-
ible lateral loads were applied to the thick concrete slab by
a hydraulic jack. The loading history with cycles versus the
displacement was illustrated in Fig. 3. Lateral displacements
increased by 1 mm (0.04 in.) every two cycles and termi-
nated when the lateral strength of specimens dropped to less
than 80% of the recorded maximum strength. The bottom
of the wall was connected to the laboratory strong floor by
prestressing rods, which aimed to prevent horizontal move-
ments between the wall base and the strong floor as well as
overturning moments. Lateral braces consisting of several

<table>
<thead>
<tr>
<th>Specimen</th>
<th>h_w/l_w</th>
<th>A_g, m² (in.²)</th>
<th>f_c', MPa (ksi)</th>
<th>f_y, MPa (ksi)</th>
<th>P/f_c' A_g, %</th>
<th>θ, deg</th>
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</thead>
<tbody>
<tr>
<td>TP0D0</td>
<td>0.67</td>
<td>0.29 (449.50)</td>
<td>35.0 (5.1)</td>
<td>335 (48.6)</td>
<td>0.72</td>
<td>0</td>
</tr>
<tr>
<td>TP0D45</td>
<td>0.67</td>
<td>0.29 (449.50)</td>
<td>36.2 (5.3)</td>
<td>335 (48.6)</td>
<td>0.70</td>
<td>45</td>
</tr>
<tr>
<td>TP5D0</td>
<td>0.67</td>
<td>0.29 (449.50)</td>
<td>35.0 (5.1)</td>
<td>335 (48.6)</td>
<td>5.65</td>
<td>0</td>
</tr>
<tr>
<td>TP5D45</td>
<td>0.67</td>
<td>0.29 (449.50)</td>
<td>36.2 (5.3)</td>
<td>335 (48.6)</td>
<td>5.46</td>
<td>45</td>
</tr>
</tbody>
</table>

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

Fig. 2—Test setup.
steel plates and load cells were attached to the reaction walls to prevent out-of-plane deformations. All specimens shared the same loading apparatus.

Numerous measuring devices were used in the test to record strains and deformations. Strain gauges were placed on both longitudinal and horizontal bars at selected locations within the specimens. 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. Sliding shear displacements were measured by a pair of LVDTs located near the bottom of wall segment.

**EXPERIMENTAL OBSERVATIONS AND RESULTS**

**Cracking pattern and failure mechanism**

Figures 5 through 8 demonstrate the development of cracks on the surface of each test specimen at three levels of drift ratios corresponding to the initial stage, the crack development stage, and the final failure stage respectively. As seen from the figures, initial shear cracks occurred at the middle and upper part of wall segment after the first run of lateral loads, which equals to a displacement of 1 mm (0.04 in.). After that, more diagonal cracks appeared, intersected with each other, and gradually spread over the web. TP5D0 with more axial loads had a steeper angle of inclined cracks on its web when compared with its counterpart. Furthermore, two surfaces of the web generally showed a similar distribution of cracks. For cracking patterns on flanges, TP0D0...
Fig. 6—Cracking patterns of TP0D45.

Fig. 7—Cracking patterns of TP5D0.
and TP5D0 with orthogonal lateral loading had most of their cracks as horizontal flexural cracks, which extended to the full length of the flange as lateral loads increased, while in TP0D45 and TP5D45 with skew lateral loading, more diagonal shear cracks appeared. As the test progressed to the failure, diagonal cracks on the web became wider and spread to the entire surfaces, causing concrete spalling at the web of some specimens. The ultimate drift ratios of specimens at failure were ranging from 1.4% to 1.8%.

Figure 9 depicts the web of TP0D0 and TP5D0 at failure. The former shared several similar characteristics with diagonal compression failure, which had been proved common in walls with large flanges. With increases in lateral displacements, inclined cracks formed by the cyclic loading intersected with each other, causing moderate concrete spalling along the diagonal strut of the web. Unlike the typical diagonal compression failure characterized by concrete crushing at the toe of the web, TP0D0 failed when the right upper corner suddenly crushed and dropped, which resulted in a drastic loss of strength.

TP5D0 failed as severe concrete spalling at the upper part of the web formed a major horizontal sliding plane. Unlike the typical sliding shear failure that usually happens near the base of the wall, the sliding was observed in the upper part of TP5D0. The force transfer mechanism was believed to be the contributing factor of this phenomenon. As stated previously, a large and stiff concrete slab was used to transfer lateral loads to wall segments. Ideally, horizontal displace-
ments 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 concentration of stress in the upper part of the web. Under the cyclic loading, more and more cracks would develop at this part, narrowing the concrete struts between the cracks. As concrete was crushed under such high stress, a weakened horizontal plane gradually formed at the upper part, which eventually led to the formation of the sliding plane. This kind of failure mode is strongly related to the stiff top slab and may be unlikely for wall in real buildings, depending on stiffness of the slabs. Nevertheless, the specimens provided useful information for behavior of squat RC walls.

As for TP0D45 and TP5D45 with skew lateral loading, both specimens were able to sustain a ductile post-peak response before failure. As the drift ratio further increased, a slight drop of shear strength was witnessed, but in general, the two specimens maintained their strength for several cycles. During this stage, the diagonal cracks became wider and slight concrete spalling was observed in the web. The drift ratio of 1.5% saw the failure of tested specimens, which was illustrated in Fig. 10. As seen in the left, the crushing of concrete in the upper left corner of the web led to a sudden drop of lateral strength to approximately 30% of the maximum, which exceeded the failure criteria. Hence, TP0D45 was deemed failed. A similar failure mechanism was also observed in TP5D45, where the specimen was unable to provide adequate lateral strength after a corner of diagonal concrete strut crushed.

**Hysteretic response**

Figure 11 illustrates the force-displacement loops of T-shaped squat walls, through which backbone envelopes and secant stiffness were extracted, as shown in Fig. 12. It was clear that specimens were able to sustain a ductile post-peak response prior to failure, except for TP0D0. Although TP0D0 had a larger maximum shear strength than TP5D0, it was evident that the specimen with more axial loads constantly had a larger lateral reaction force at the same drift.
ratio, which was consistent with previous findings. A similar trend could also be traced in the group with skew lateral loading. When it comes to the deformation capacity, it was noticeable that TP0D0 outperformed its counterpart (1.7% compared to 1.4%). However, the specimens with skew lateral loading demonstrated nearly the same capability.

Curves of secant stiffness versus drift ratio are presented in Fig. 12. As expected, all specimens experienced a decline in stiffness throughout the test. At the initial stage, the stiffness of each specimen decreased rapidly, in which the most dramatic drop occurred in TP5D0 and TP5D45. With the increase of drift ratio, the secant stiffness of each specimen further declined and remained at a comparatively low value at the final stage. Moreover, axial loads could be seen as an active factor in increasing the secant stiffness. Its contribution could be attributed to the fact that axial loads constrained the diagonal cracks and limited the development of the sliding shear, enhancing the reaction at the specific drift ratio.

**Displacement component**

Top deformations of structural walls mainly consist of three components: flexural deformation, shear deformation, and sliding deformation. Figure 13 illustrates the ratios of these components to the total top displacement during the test, in which an unaccounted portion was also presented because the summation of the other three could not always reach a perfect 100%.
The shear deformation was found to dominate the response because it accounted for the largest portion of total deformation in most of the test specimens. A maximum value of 64% was witnessed in TP5D0 in the negative loading direction. It was evident that the portion of the shear deformation generally increased during all tests, while declining trends were recorded in the flexural deformation. It was also found that specimens with skew lateral loading had a slightly less shear deformation as compared with their counterparts loaded orthogonally. This could be attributed to the fact that the principal shear force was transferred by multiple segments in the former, while only one segment mainly functioned in the latter, which was in accordance with the findings of Zhang and Li. The shear displacement also experienced larger growth in specimens with orthogonal lateral loading. Regarding the influence of axial loads, a slight growth of the shear deformation was observed in the orthogonally loaded specimens, whereas the impact was negligible in specimens with skew lateral loading. The unaccounted part varied slightly, but its value remained relatively small.

As for the composition of the sliding shear displacement, a continuous growth was observed in all specimens, with values rising from 3% to 5% to 8% to 15%. A more rapid increase was found in specimens with no additional axial loads, which was reasonable because axial loads could effectively limit the width of cracks, thereby slowing the formation of horizontal cracks planes. Moreover, there was no significant variation between the portions of the sliding shear displacement in specimens with orthogonal lateral loading and specimens with skew lateral loading, indicating that the change of loading direction did not play a favorable role in the control of the sliding shear.

Strain profile

The strain variations along the bottom of the longitudinal reinforcement in the flange are presented in Fig. 14. Yielding was found during the tests in all specimens. A significant shear lag effect could be detected in TP0D0 and TP5D0. This phenomenon was more evident in TP5D0, in which strains of the outmost longitudinal reinforcement were almost zero when the flange was in tension, suggesting that the outer part of the flange was hardly involved in withstanding lateral loads. A similar pattern of strain distributions was also observed in the experiment of T-shaped slender walls conducted by Thomsen and Wallace and Brueggen et al. However, unlike the observations in these tests, where all longitudinal reinforcement yielded at the maximum lateral strength, in this study, only a small amount of the reinforcing bar experienced yielding in this test. In specimens with skew lateral loading, the shear lag effect was less obvious but still could be traced as the tensile strains in longitudinal bars showed an increasing trend toward the flange tip. However, it should be noticed that the maximum strain occurred 240 mm (9.45 in.) from the flange tip, which was in accordance with the results of section analyses.
Table 2—Equations for shear strength of reinforced concrete squat walls

1. Section 11.5 of ACI 318-14\textsuperscript{18}
\[ V_{n} = V_{c} + V_{r} \geq 0.83 \sqrt{f_{c}^{'}} h_{d} \text{ (MPa)} \]
\[ V_{c} = 0.27 \sqrt{f_{c}^{'}} h_{d} + N_{d}/A_{n} \text{ or } V_{c} = [0.055 \sqrt{f_{c}^{'}} + (0.1 \alpha \sqrt{f_{c}^{'}} + 0.2 N_{d}/l_{h})/(M_{f} V_{c} - l_{h})] h_{d} \]
\[ V_{r} = A_{f} d \ell/s \]

2. Section 18.10 of ACI 318-14\textsuperscript{18}
\[ V_{n} = A_{s} (0.25 \sqrt{f_{c}^{'}} + \rho_{o} f_{y}^{'}) \leq 0.83 A_{n} \sqrt{f_{c}^{'}} \text{ (MPa)} \]
\( \alpha \) is a coefficient: 0.25 for \( h_{d}/l_{h} \leq 1.5 \), 0.17 for \( h_{d}/l_{h} \geq 2.0 \), and varies linearly between 0.25 and 0.17 for \( h_{d}/l_{h} \) between 1.5 and 2.0

3. ASCE 43-05\textsuperscript{24}
\[ V_{n} = \phi d_{h}(8.3 \sqrt{f_{c}^{'}} - 3.4 \sqrt{l_{w} (h_{d}/l_{h} - 0.5) + N_{d}/A_{n} l_{w} + \rho_{o} f_{y}^{'})} \leq 0.2 \sqrt{f_{c}^{'}} d_{w} \]
\( \rho_{o} = \rho_{f} + B_{f} \rho_{a} \) and \( A \) and \( B \) are constants related to shear-span ratio.

4. Barda et al. 43-05\textsuperscript{18}
\[ V_{n} = (8.3 \sqrt{f_{c}^{'}} - 2.5 \sqrt{l_{w} (h_{d}/l_{h}) + N_{d}/A_{n} l_{w} + \rho_{o} f_{y}^{'}}) h_{d} \text{ (lb)} \]

5. Wood\textsuperscript{15}
\[ 6 A_{s} \sqrt{f_{c}^{'}} \leq V_{n} = A_{f} d \ell/s \leq 10 A_{n} \sqrt{f_{c}^{'}} \text{ (lb)} \]

6. Hwang et al.\textsuperscript{26}
\[ V_{n} = (K_{s} + K_{c} - 1) \xi^{2}/A_{n} \cos \theta \text{ (MPa)} \]
\( \xi \approx 3.5 \sqrt{f_{c}^{'}} \leq 0.5 \), \( A_{w} = a_{w} \times t_{w} \times \sin \theta = a_{w} = (0.25 + 0.85 V_{w}^{'}/l_{w}) \), \( \theta = \tan^{-1}(h_{d}/l_{h}) \)
\( K_{s} \) and \( K_{c} \) are indexes of horizontal tie and vertical tie, respectively

7. Gulec and Whittaker\textsuperscript{14}
\[ V_{n} = (0.04 f_{w}^{'}/A_{w} + 0.40 f_{w}^{'}/A_{w} + 0.15 f_{w}^{'}/A_{w} + 0.35 P_{w}) \sqrt{h_{d}/l_{h}} \leq 15 \sqrt{f_{c}^{'}} A_{s} \text{ (lb)} \]

Table 3—Ratios of predicted to measured shear strength using different sets of equations

<table>
<thead>
<tr>
<th>Specimens</th>
<th>ACI 318-14\textsuperscript{18} §11.5</th>
<th>ACI 318-14\textsuperscript{18} §18.10</th>
<th>ASCE 43-05\textsuperscript{24}</th>
<th>Barda et al.\textsuperscript{23}</th>
<th>Wood\textsuperscript{15}</th>
<th>Hwang et al.\textsuperscript{26}</th>
<th>Gulec and Whittaker\textsuperscript{14}</th>
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<tr>
<td>TP0D0</td>
<td>0.58</td>
<td>0.67</td>
<td>0.99</td>
<td>1.32</td>
<td>0.56</td>
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<td>TP0D45</td>
<td>0.57</td>
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<td>0.98</td>
<td>1.30</td>
<td>0.55</td>
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<td>1.13</td>
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<tr>
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<td>1.61</td>
<td>0.63</td>
<td>1.54</td>
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<td>TP5D45</td>
<td>0.58</td>
<td>0.57</td>
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<td>0.47</td>
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<td>Mean</td>
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<td>0.665</td>
<td>1.001</td>
<td>1.361</td>
<td>0.549</td>
<td>1.209</td>
<td>1.244</td>
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<td>Median</td>
<td>0.580</td>
<td>0.669</td>
<td>1.986</td>
<td>1.311</td>
<td>0.552</td>
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<td>SD</td>
<td>0.097</td>
<td>0.077</td>
<td>0.115</td>
<td>0.174</td>
<td>0.065</td>
<td>0.230</td>
<td>0.206</td>
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<tr>
<td>COV</td>
<td>0.156</td>
<td>0.116</td>
<td>0.115</td>
<td>0.128</td>
<td>0.118</td>
<td>0.190</td>
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<tr>
<td>Overprediction, %</td>
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<td>0.0</td>
<td>25.0</td>
<td>100.0</td>
<td>0.0</td>
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</table>

Notes: SD is standard deviation; COV is coefficient of variation.

**DISCUSSIONS OF EXPERIMENTAL RESULTS**

**Evaluation of shear strength**

Shear strength plays a key role in the design of RC structural walls, as it is closely related to the performance of walls under earthquakes. To ensure accurate evaluations, a significant number of studies had been done using both experiments\textsuperscript{14-16} and analytical models.\textsuperscript{21,22} However, as mentioned earlier, limited information was provided on the design of non-rectangular squat RC walls.\textsuperscript{23}

Three major building codes could be adopted to predict the nominal shear strength of non-rectangular squat walls, which were Section 11.5 of ACI 318-14 for structural walls, Section 18.10 of ACI 318-14 for earthquake-resistant structures, and ASCE 43-05\textsuperscript{24} for the seismic design of structures in nuclear facilities. Besides these code provisions, Barda et al.,\textsuperscript{25} Wood,\textsuperscript{15} Hwang et al.,\textsuperscript{26} and Gulec and Whittaker,\textsuperscript{14} and Gulec et al.\textsuperscript{18} also proposed relevant equations to predict the shear strength of squat RC walls, which are demonstrated in Table 2.

Table 3 presents the statistics for ratios of predicted to experimental peak shear strength using aforementioned methods. It was evident that significant scatter existed in the predictions of the peak shear strength of T-shaped specimens. Equations from Section 11.5 of ACI 318-14\textsuperscript{18} provided an underestimation of shear strength. This was because they were developed for rectangular walls, where the influence of large flanges in T-shaped walls was not reflected. Only transverse reinforcement in the web was assumed to withstand shear force in Section 11.5 of ACI 318-14, which was in contradiction with the findings of Lefas et al.\textsuperscript{23} Also, Wood\textsuperscript{15} had shown that, compared with horizontal reinforcement, the shear strength of squat walls was more dependent on longitudinal web reinforcement. Section 18.10 of ACI 318-14, based on the modified truss analogy proposed by Wood,\textsuperscript{15} had slightly better predictions than Section 11.5. However, this equation still underestimated the shear strength, mainly due to insufficient parameters it included, such as axial loads and longitudinal reinforcement. Unlike
equations in ACI 318-14, design procedures in ASCE 43-05 overestimated the shear strength of specimens. The same phenomenon was also witnessed in the equation provided by Barda et al.,\textsuperscript{25} which had similar function forms as those in ASCE 43-05. These overestimations were also observed by Gulec and Whittaker\textsuperscript{14} and Kassem\textsuperscript{28} in their evaluations of shear strength equations using a comprehensive database of flanged walls. The set of equations proposed by Hwang et al.,\textsuperscript{26} was analytically reasonable, but the effect of flange was not considered in the diagonal mechanism, the vertical mechanism, or the horizontal mechanism. Gulec and Whittaker\textsuperscript{14} used nonlinear regression to determine the coefficient of influential parameters and proposed their equation specifically for flanged shear walls. The equation incorporated the contribution of the flange and yielded reasonable results in terms of mean value and COV, which was considered superior to other sets of equations herein. Deficiencies were also found in their equation, as it overestimated the shear strength by assuming all longitudinal reinforcement in the flange were effective in resisting lateral loads.

The equations listed in the table were developed for walls under lateral loading parallel to the web, which would yield an equivalent strength for walls under lateral loading from different directions. However, specimens with skew lateral loading processed larger lateral resistances than their counterparts. Hence, it was inappropriate to invariably apply the equations to walls under nonprincipal bending action. One simple solution was to separate the web and flange segments, calculate the individual shear strength, and then sum them up by projecting the vector of strength to the axis of loading direction. As for TP0D45, the modified approach would improve the accuracy of predicted result from 0.67 to 0.91 using the expression in Section 18.10 of ACI 318-14. The force transfer mechanism between wall segments was rather intricate, thus new analytical models should be proposed comprising the effect of flanges and providing a valid mechanical base for the development of equations for the shear strength of T-shaped squat walls.

**Influence of lateral loading direction**

Traditional rectangular walls are designed for lateral loading in the principal direction. However, in reality, the seismic wave can reach structures from any direction, which necessitates studies on behaviors of walls under various lateral loading directions. Unlike rectangular walls, nonrectangular walls naturally possess the capacity to resist lateral loads in both transverse and the longitudinal directions. Thus, during an earthquake, their walls are subjected to biaxial bending action, where the contribution of lateral resistance from different wall segments could be rather complex. TP0D45 and TP5D45, as presented previously, solely provided information regarding loading direction 45 degrees from the web. To further investigate the influence of lateral loading direction, finite element analyses were carried out on T-shaped RC squat walls under lateral loading from various directions, as illustrated in Fig. 15.

A widely-used commercial nonlinear finite element software TNO Diana\textsuperscript{29} was adopted in this study, the capability of which has been proved by many researchers.\textsuperscript{2,11,25} A 20-node iso-parametric solid brick element was chosen to model concrete. The reinforcing bars were modeled as the embedded reinforcement, with the assumption of a perfect bond between the reinforcement and surrounding concrete. The total strain rotating crack model was adopted to account for the nonlinear behaviors of concrete. The reduction of compressive strength of concrete brought by tensile strains and accompanying cracks was explained in the model of Vecchio and Collins.\textsuperscript{30} The impact of lateral confinements on the compressive stress-strain curve of concrete was defined based on the model proposed by Selby et al.\textsuperscript{31} The compressive behavior of concrete was predefined using the parabolic curve, and the tensile softening was simulated by adopting the model proposed by Hordijk et al.\textsuperscript{29} Von Mises yield criteria were applied to the reinforcement, and stress-strain curves derived from reinforcing bar testing were used to model steel. The aforementioned finite element methods were validated with experimental results, as seen in Fig. 11. In general, it could be concluded that the introduced numeric models can successfully simulate the behavior of T-shaped RC squat walls under lateral cyclic loading. Nevertheless, it was witnessed in TP5D0 that the finite element model overestimated the maximum shear strength by approximately 11%, which could be attributed to the severe concrete spalling at the upper part of web that impaired the lateral resistance. Additionally, in the group without extra axial loads, it was found the numeric model possessed larger secant stiffness at the initial stage, but as the test progressed, the discrepancy gradually decreased.

Figure 16 illustrates the peak shear strength of T-shaped squat walls under lateral loads from different directions. The blue and red curves, labelled as ALR = 0.72% and ALR = 5.65%, represented T-shaped squat walls subjected to axial...
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Table 4—Ratios of test-derived base moments and tension forces in flange to those obtained from section analyses

<table>
<thead>
<tr>
<th></th>
<th>TP0D45</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moment</td>
<td>Tension force</td>
<td>Moment</td>
</tr>
<tr>
<td>500</td>
<td>0.66</td>
<td>0.68</td>
<td>0.93</td>
</tr>
<tr>
<td>1000</td>
<td>0.71</td>
<td>0.73</td>
<td>0.91</td>
</tr>
<tr>
<td>1500</td>
<td>0.73</td>
<td>0.76</td>
<td>1.00</td>
</tr>
<tr>
<td>2000</td>
<td>0.66</td>
<td>0.73</td>
<td>1.09</td>
</tr>
<tr>
<td>2500</td>
<td>0.75</td>
<td>0.80</td>
<td>1.07</td>
</tr>
<tr>
<td>3000</td>
<td>0.74</td>
<td>0.82</td>
<td>1.11</td>
</tr>
</tbody>
</table>

Fig. 16—Maximum resistance of T-shaped walls under lateral loading from various directions. (Note: 1 kN = 0.225 kip.)

Shear lag effect

A significant shear lag effect was discovered in some specimens, which was characterized by nonlinear distributions of stresses in the flange and commonly represented by the effective flange width in building codes. To investigate the shear lag effect in the flange of test specimens, comparisons of base moments and tension forces in the flange were made between measured data and results obtained from the nonlinear section analyses tool Xtract. As shown in Table 4, it was clear that the nonlinear section analyses overestimated the base moments as well as the tension forces of test specimens, due to the underlying assumption that the plane section remained plane and the consequent neglect of shear lag effects, which served as proof that the shear lag effect should be carefully considered when calculating the lateral strength of T-shaped squat walls.

The shear lag effects were found to be affected by the loading directions, drift ratios, and axial loads. The shear lag effect was more distinct on specimens with symmetrical loading direction. For instance, the average of ratios of experimental-to-predicted moments was 0.54 for symmetrically-loaded TP5D0, while its counterpart TP5D45 with skew lateral loading only had 0.83. The influences of the shear lag effect were found to be most distinct in all specimens at initial loading stage. As the drift ratio rose, the effect of shear lag on the load capacity gradually diminished when more longitudinal reinforcing bars in the flange reached the yielding point, which was in accordance with the finding of Hassan and El-Tawil. This is due to that the large differences in strains could hardly lead to the equivalent disparity of stress in steel after the longitudinal reinforcing bar in the flange yielded, resulting in a slower growth of moment resisted by the section. It was observed that additional axial loads generally increased the impact of shear lag effects.

Effective stiffness

The effective stiffness, known to be an important factor in the displacement-based design, plays a pivotal role in the seismic performance of RC structures, which can significantly affect the design of displacements, ductility factors, and internal forces. Building codes usually employ reduction factors of gross moment of inertia to account for the influence of the loss of flexural and shear rigidities. In ACI
318-14, the reduction factors 0.35 and 0.70 are recommended for cracked and uncracked concrete. As specified in ASCE 43-05, no reduction is recommended for uncracked concrete and 0.5 is recommended for cracked concrete.

Many researchers\textsuperscript{34,35} have found that this simplification overestimated the effective stiffness of squat walls. Priestley and Kowalsky\textsuperscript{27} and Paulay et al.\textsuperscript{36} suggested that the effective stiffness of flexural walls should be closely related to axial loads and reinforcement contents as

$$\frac{I}{I_g} = \frac{100}{f_y} + \frac{P_e}{f_y'A_g}$$

(1)

To account for the effect of shear deformations on wall stiffness, the following expressions were also developed for structural walls with aspect ratios less than 4

$$I_w = \frac{I_o}{1.2 + F}$$

(2)

$$F = \frac{30I}{b_w h_w I_w}$$

(3)

Li and Xiang\textsuperscript{34} noticed this imperfection and proposed an equation specifically for squat walls as shown in the following

$$\frac{I}{I_g} = 0.19 \left( \frac{100}{f_y} + \frac{N}{f_y'A_g} \right) \left( 0.53 + 0.37 \frac{b_w}{L_w} + 0.31 \frac{h^2_w}{L_w} \right)$$

(4)

The evaluations of the aforementioned two sets of equations using data obtained from this study are shown in Table 5. Generally, both methods overestimated the test results, which is understandable. The large flanges could significantly increase the moment of inertia of gross section. Due to the shear lag effect, it was inappropriate to adopt the moment of inertia of the entire uncracked section in estimating the effective stiffness of T-shaped walls. Further research is required to investigate the effective stiffness of nonrectangular squat walls.

**SUMMARY AND CONCLUSIONS**

The experimental results of four T-shaped RC squat walls were presented and discussed first. After that, nonlinear section analyses and finite element modeling were performed to investigate their seismic performance from various aspects. Comparisons were also conducted between slender and squat T-shaped walls to highlight the influence of shear span ratios. Primary conclusions are enumerated as follows:

1. TP0D0 failed with moderate concrete spalling along the diagonal of the web, which shared similar characteristics with the diagonal compression failure. The intersection of inclined 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 TP5D0. This kind of failure mechanism is strongly related to the stiff concrete head and may be unlikely in real structures. The other two specimens with skew lateral loading were able to sustain a ductile post-peak response for few cycles, and failed with a dramatic drop of lateral strength when concrete in upper left corner of the web suddenly crushed.

2. The shear deformation, which dominated the response of walls, was found more evident in specimens with orthogonal lateral loading, with a maximum value of 64% recorded in TP5D0. Sustained slight increases of the sliding shear displacement from 3% to 5% to 8% to 15% were observed in the test, in which a more rapid growth was found in the specimen with lower axial loads.

3. Significant scatters existed in the predictions of peak shear strength of tested T-shaped squat walls using equations from building codes and different researchers. Most equations underestimated the peak shear strength of T-shaped squat walls because the effect of large flanges was not incorporated. Moreover, it was observed that specimens with skew lateral loads processed larger lateral strengths than their counterparts with orthogonal lateral loads. Thus, it was inappropriate to apply those equations to walls subjected to nonprincipal bending action.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial load ratio, %</th>
<th>Direction of lateral loads</th>
<th>Loading direction</th>
<th>(\frac{I}{I_g}) (\frac{I}{I_g}) (\frac{I}{I_g})</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eq. (1) to (3)</td>
<td>Eq. (4)</td>
<td>Test</td>
<td>(1)/(3)</td>
<td>(2)/(3)</td>
</tr>
<tr>
<td>TP0D0</td>
<td>0.72 Orthogonal</td>
<td>Positive</td>
<td>0.066</td>
<td>0.033</td>
<td>0.016</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>0.066</td>
<td>0.033</td>
<td>0.020</td>
<td>3.34</td>
</tr>
<tr>
<td>TP0D45</td>
<td>0.70 Skew</td>
<td>Positive</td>
<td>0.066</td>
<td>0.033</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>0.066</td>
<td>0.033</td>
<td>0.029</td>
<td>2.27</td>
</tr>
<tr>
<td>TP5D0</td>
<td>5.65 Orthogonal</td>
<td>Positive</td>
<td>0.072</td>
<td>0.041</td>
<td>0.022</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>0.072</td>
<td>0.041</td>
<td>0.028</td>
<td>2.56</td>
</tr>
<tr>
<td>TP5D45</td>
<td>5.46 Skew</td>
<td>Positive</td>
<td>0.073</td>
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<tr>
<td></td>
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<td>0.073</td>
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<td>0.036</td>
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<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>2.807</td>
<td>1.488</td>
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</tr>
<tr>
<td>SD</td>
<td></td>
<td></td>
<td>0.681</td>
<td>0.331</td>
<td></td>
</tr>
</tbody>
</table>
4. From finite element analyses, it was revealed that the peak shear strength of T-shaped squat walls varied at different lateral loading directions. The maximum peak shear strength was found when lateral loads were 40 degrees from the web and a reduction as much as 30% was observed as the direction of loading changed. Declines of 20% and 19% of the strength were recorded in squat and slender T-shaped walls, respectively, as the loading direction changed from parallel to the web to perpendicular to the web. Hence, it was necessary to consider the influence of the lateral loading direction in design procedures.

5. A significant shear lag effect was detected in some specimens, resulting in only 70% of the tension force, as expected, being developed in the flange of orthogonally loaded specimens. It was found that additional axial loads increased the impact of the shear lag effect while the increase of lateral displacements reduced the influence of the shear lag effect, which was in consistent with previous findings in T-shaped slender walls. Similar to the slender walls, T-shaped squat walls were more affected by the shear lag effect.

6. The mean value of measured stiffness ratio \( I_s/I_g \) was 0.026 for test specimens, which was overestimated by methods available in the literature. As corroborated in the T-shaped slender walls, the increase of axial loads brought growth on the effective stiffness. Also, specimens with skew lateral loading had greater effective stiffness than specimens with orthogonal lateral loading.

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**NOTATION**

- \( A_g \) = gross section area
- \( b_w \) = depth of wall
- \( d \) = length of wall section
- \( E \) = elastic modulus of concrete
- \( f'_c \) = compressive strength of concrete
- \( f_y \) = yield strength of reinforcement
- \( h_w \) = height of wall
- \( I_e \) = aspect ratio
- \( I_g \) = effective moment of inertia
- \( I_w \) = moment of inertia of gross section
- \( l_w \) = length of wall
- \( l_g \) = Length of wall
- \( N \) = axial load
- \( P/L_e \) = axial load ratio
- \( \theta \) = angle between lateral loading and web segment
- \( \rho_h \) = horizontal reinforcement ratio
- \( \rho_l \) = longitudinal reinforcement ratio

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