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Seismic behaviour of non-rectangular structural RC wall in the weak axis

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Structural reinforced concrete (RC) walls can have very different lengths and widths. Previous investigations have focused on the seismic properties of structural RC walls in the strong axis in which the length of the wall is greater than the width of the wall. The seismic behaviour of structural RC walls in the weak axis, where the wall length is less than its width, is insufficiently investigated. This paper reports on experimental studies on the seismic behaviour of two non-rectangular RC walls loaded in the weak axis. The experimental results are presented in terms of cracking patterns, failure mechanisms, hysteresis responses, the component of deformation and the distributions of vertical strains in the flange. The experimental results are discussed, and emphasise the accuracy of existing design methods for non-rectangular RC walls in the weak axis, including design methods for the stiffness, the shear lag effect and the shear strength.

Notation

A_{cv}	gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered
A_g	gross section area
A_l	area of the vertical reinforcement
A_v	area of shear reinforcement spacing
b_w	depth of wall
d	distance from extreme compression fibre to centroid of longitudinal tension reinforcement
E_c	elastic modulus of concrete
f'_c	compressive strength of concrete
f_y	yield strength of reinforcement
h_w	height of wall
h_w/l_w	shear span ratio
I_g	moment of inertia of gross section
I_w	effective moment of inertia
l_w	length of wall
M_u	factored moment at section
$P_u/f'_c A_g$	axial load ratio
P_u	axial load
s	spacing of horizontal reinforcement
t	thickness of wall segment
V_u	factored shear force at section
θ	angle between lateral loading and web segment
ρ_t	ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement

Introduction

Structural reinforced concrete (RC) walls are an important component in the seismic load resistance of RC buildings due to their large in-plane stiffness. Such walls can have non-rectangular sections in order to further increase their lateral

stiffness or due to the architectural plan. Sometimes, the width (the projective depth of the wall perpendicular to the assumed lateral loading direction) of non-rectangular RC walls can be significantly less than their length (the projective depth of the wall along the assumed lateral loading direction), as shown in Figure 1. For rectangular RC walls, design engineers often ignore the lateral stiffness of the wall in the shorter direction. However, for non-rectangular RC walls, considerations and calculations of the lateral load capacity and deformation capacity may be required for various loading directions. For the non-rectangular RC walls shown in Figure 1, the length of the wall would be significantly shorter than the width when the lateral load is applied in the transverse direction. Beyer *et al.* (2008), Lowes *et al.* (2013), Aaleti *et al.* (2013) and Zhang and Li (2016a) conducted investigations on the seismic behaviour of non-rectangular RC walls considering different loading directions, but these studies mainly focused on walls where the length and width were similar. Previous research has indicated that the wall width could affect the stiffness (Zhang and Li, 2016a), the shear lag effect (Hassan and El-Tawil, 2003; Pantazopoulou and Moehle, 1990; Zhang and Li, 2016b) and the shear strength (Barda *et al.*, 1977; Gulec and Whittaker, 2011; Wood, 1990) of non-rectangular RC walls. Therefore, the conclusions drawn from previous investigations on non-rectangular RC walls may need further evaluation for walls loaded in the weak axis in which the length is significantly less than the width.

The work reported here aimed to investigate the seismic behaviour of structural RC walls in the weak axis. The walls were designed to have a large lateral stiffness for resisting lateral load in the strong axis and were shorter in length compared with the width when loaded in the weak axis. Experiments were conducted on two specimens, and the behaviour of the non-rectangular RC walls in the weak axis was further evaluated in terms of the stiffness, the shear lag effect and the shear strength.

Figure 1. Common sections of structural walls in buildings

Experimental programme

Specimen details

The experimental programme included the testing of two specimens, named SLW and STW. Specimen SLW was an L-shaped wall, as shown in Figure 2(a) and STW was a T-shaped wall, as shown in Figure 2(b). The walls had the same height, length and width. The vertical reinforcing ratios of the two walls were slightly different, with 1.65% for SLW and 1.57% for STW.

Figure 2. Design of specimens (dimensions in mm)

Table 1. Experimental parameters and principal results of walls tested

Specimen	Section: mm	h_w : mm	$P_u/f_c A_g$	A_l : mm ²	s : mm	f_c : MPa	Vertical reinforcement ratio: %		
							Overall	Web	Flange
SLW	600 × 900 × 100	2534	0.10	2316	150	35.2	1.65	2.16	1.13
STW	600 × 900 × 100	2534	0.10	2204	150	31.7	1.57	2.16	1.00

Figure 3. Experimental setup: 1, hydraulic jack for lateral load; 2, loading head; 3, actuator for axial load; 4, specimen

The two walls were shorter in length (0.6 m) compared with their width (0.9 m). The walls were connected with a concrete base cast with the body of the wall. A summary of the design parameters of the walls is given in Table 1. The specimens were cast using grade 40 concrete. The average compressive strengths (f_c) of the concrete measured at the date of testing are also given in Table 1. The $\varnothing 12$ and $\varnothing 6$ reinforcing bars used in the specimens had characteristic yield strengths of 545 MPa and 565 MPa, respectively. The final strengths of these bars were taken as 675 MPa and 705 MPa.

Loading apparatus

The specimens were installed in the loading apparatus shown in Figure 3. The axial load was applied by two vertical actuators, as shown in the figure. These vertical actuators ensured the free-rotating boundary condition of the specimen by maintaining zero moment at the top of the wall. Horizontal cyclic loading was applied by a horizontal hydraulic jack. In addition to the jack, two lateral braces were installed in the lateral direction of the specimen to ensure that the top displacement was in the in-plane direction. The actuators, hydraulic jack and lateral braces were all pin-connected at both ends. The bottom of the wall was connected to a laboratory strong floor by pre-stressing rods, with the aim of preventing horizontal movement between the wall base and the strong floor as well as overturning movement.

Instrumentation

Besides linear variable differential transformers (LVDTs) and load cells inside the vertical actuators, load cells were also installed adjacent to the horizontal jack to measure the lateral load. Additionally, two load cells were attached to the lateral braces to measure the reaction of the wall perpendicular to the loading direction. Displacement measurements were taken by means of string potentiometers for measuring the top displacement, and LVDTs were attached to the body of the wall to distinguish the different components of deformations. A schematic illustration of the layout of the LVDTs is given in Figure 4. For SLW, the LVDTs were installed on both segments of the wall whereas, due to the symmetry of the section, the LVDTs were only installed on one flange segment of the wall for STW. Strain gauges were mounted on longitudinal, horizontal and transverse reinforcements at critical locations to capture the strain readings.

Loading procedure

The axial load was applied to the specimen before the horizontal load. Thereafter, a constant axial load of $0.1f_c A_g$ was maintained for both specimens. The horizontal load was initially force-controlled, with a peak load of 20 kN for the first two cycles. The peak lateral load was increased by 20 kN for each two cycles in the following load reversals until the lateral displacement of the wall reached 10 mm. The lateral load was then displacement-controlled, with an increment of 10 mm every two cycles. The loading procedure is illustrated in Figure 5. The test was terminated when the wall lost 20% of its lateral strength.

Experimental results

Crack patterns and failure modes

Figure 6 shows the observed cracking patterns of the specimens at the final stage. The cracking patterns in the two specimens shared many similarities. Cracks were distributed over the web of the wall, induced by the reversed cyclic loadings. The upper part of the wall featured diagonal cracks, induced by the shear force. The lower part of the wall showed fan-like cracks in which the diagonal crack at the middle of the web of the wall connected with the horizontal cracks at the edges. Also, the direction of the diagonal crack changed at the base of the wall and the angle increased with height. For the flange, cracks were only observed in the flange in the tension loading direction for both specimens. The cracks in the flange were a mixture of horizontal and diagonal cracks. The diagonal

Figure 4. Layout of LVDTs in the specimens (dimensions in mm)

Figure 5. Lateral load imposed to the specimens

cracks in the flange generally propagated upwards the further they were away from the web.

The failure modes of the two specimens were also found to be similar (Figure 7). For both specimens, the final failure was as a consequence of web crushing at the tip of the web. The crushing started at the end of the compressive strut in the wall and rapidly progressed to other areas. The strength of the wall dropped rapidly as the crushed area enlarged.

Load–displacement curves

Figure 8 shows the load–displacement curves of the two specimens. A more detailed comparison of the load–displacement responses of the two specimens is given in Table 2. The table shows that the first-yield lateral load and displacement differed slightly in the two specimens and in opposite loading directions. The yield force and displacement in the flange in the tension loading direction were found to be larger than those in the compression loading direction for both specimens. The

Figure 6. Cracking patterns of web and flange of SLW ((a) and (b)) and web and flange of STW ((c) and (d))

first-yield lateral force of SLW in the flange in the tension direction was 54% larger than that in the opposite direction. STW yielded later than SLW, with a yielding displacement of 9.1 mm in the flange in the tension direction and 8.6 mm in the flange in the compression direction. For STW, the corresponding displacements were 8.6 mm and 6.7 mm. Similar tendencies were observed for the lateral force. For SLW, the maximum lateral strengths in both loading directions were similar, 176 kN for the flange in the compression loading

direction and 163 kN for the flange in the tension loading direction. For STW, the maximum lateral strength in the flange in the tension direction was larger than that in the compression direction (163 kN compared with 113 kN). Both specimens suffered strength degradation and loss of lateral strength when the lateral displacement was greater than 50 mm. The deformation capacity of SLW was slightly better than that of STW. SLW was able to sustain cycles with 60 mm deformation, whereas STW failed at the second cycle with 50 mm displacement.

Components of top deformation

The components of top deformations were calculated based on the LVDTs attached to the wall. The components are reported in Figure 9. Specifically, the flexural deformations of the wall were derived by accumulating the curvature of the wall measured by the vertical chains of the LVDTs over the height of the wall. The shear deformations were captured by the diagonal string potentiometer. Calculation of the shear deformation was based on readings from the diagonal LVDTs and the vertical LVDTs measuring curvatures of specimens, as proposed by Hiraishi (1984). The sliding deformation was found to be less than 1% of the total deformation for the two specimens. This component of deformation is omitted in Figure 9. The sum of the shear deformation and the flexural deformation did not always equal the total deformation, and the differences are reported as 'error' in Figure 9.

The portion of the shear deformation was found to be quite significant for both specimens. For SLW, the portion of shear deformation was around 15% in the flange in the tension direction and around 5% in the flange in the compression direction. The portion of shear deformation was even larger for STW – around 30% in the flange in the tension direction and 15% in the flange in the compression direction. These portions are rather significant, as the shear span ratio of the two specimens was 4.22, so they can be considered as slender walls. The portions of the shear deformation were also quite different for the L-shaped and T-shaped walls despite the fact they had similar configurations. This may relate to the unsymmetrical section of SLW.

Strain profile in the flange

Figure 10 illustrates the strains as measured by strain gauges attached to the vertical reinforcing bars in the flange. The strains are given corresponding to lateral drift ratios of 0.3%, 0.8%, 1.2%, 1.6% and 2.1%. As shown in the figure, the distribution of the vertical strains in the specimens was significantly affected by the shear lag effect. The influence can be seen in the flange in both the tension direction and the compression direction. For SLW at a lateral drift ratio of 1.6%, the vertical strain of the bar adjacent to the web was 150% larger than that in the end of the flange when the flange was in compression and 310% larger while the flange was in tension. For STW at

Figure 7. Failure of specimen SLW ((a) and (b)) and specimen STW ((c) and (d))

the same drift ratio, the shear lag effect caused the vertical strains to be 104% and 50% larger in the bar adjacent to the web when the flange was in compression and in tension, respectively. More specifically, the shear lag effect related to yielding of the reinforcing bars. For SLW, in the flange in the tension direction, the vertical bars at the toe of the flange did not yield until a lateral drift ratio of 1.2%, while the bar adjacent to the web yielded at 0.8%.

Design implications

Stiffness of non-rectangular RC walls in the weak axis

The stiffness of an RC member is important in design as it often determines the seismic forces on the building and the distribution of forces in different structural members. As previous investigations have generally focused on walls of length similar to or larger than the width, the accuracy of existing methods was assessed using the experimental data obtained in the current study. Table 3 summarises the measured stiffness properties from the tests, including the effective stiffness and the first-yield displacement. In conjunction with the experimentally derived values, the methods proposed by Paulay and Priestley (1992) and Zhang and Li (2016a) for estimating the effective stiffness of a wall and the method proposed by Priestley and Kowalsky (1998) for estimating the first-yield lateral displacement of the wall are also included in the table. For the effective stiffness, the ratio I_w/I_g of the tested specimens ranged from 0.36 to 0.46. The method proposed by Paulay and Priestley (1992) underestimated the effective stiffness of

the specimens by 30–50%. The equation proposed by Zhang and Li (2016a) for the T-shaped walls estimated the effective stiffness more accurately, with a slight overestimation of 10%. With regards to the first-yield displacement, the estimated values were within 35% of the experimentally derived values. This is believed to be an acceptable level of accuracy, as the equation was proposed for the primary design stage.

The yield displacement of a structural wall generally decreases with the length of the wall. There have been concerns in the research community (Mergos and Beyer, 2014) that when a longer wall yields while the shorter wall does not in a structural system, a distinctly larger portion of shear force will be carried by the shorter wall. A more direct comparison of the stiffness of the wall and its impact on behaviour is given in Figure 11, which compares the pushover curves of the wall in the weak axis and the strong axis (perpendicular to the top displacement imposed in the tests). Specifically, the pushover curves in the strong axis were derived from finite-element analysis based on a similar approach to that presented by Zhang and Li (2016b), while the pushover curves for the weak axis were derived from the peak displacements from the cyclic experimental curves. Figure 11 shows the portions of the lateral stiffness of the wall in the weak axis compared with the lateral stiffness in the strong axis. As the plots illustrate, the lateral strengths of the wall in the weak axis account for at least 20% of those in the strong axis. The portion varies depending on the sectional shape and loading direction. Additionally, the portions are also different at different lateral

displacements. For SLW, the portion was found to be around 50% for the flange in the compression direction and 30% for the flange in the tension direction. The portion was found to increase with increasing lateral displacement. For STW, the

portion was significantly larger, with an average value of around 50% for the flange in the compression direction and 70% in the flange in the tension direction. The influence of lateral displacement was also found to be more complicated, as the portion decreased before the wall yielded but remained relatively constant after a lateral displacement of 10 mm.

Figure 8. Hysteresis loops of the tested specimens

Shear lag effect

It was generally found that the strength and stiffness of non-rectangular RC walls are significantly affected by the shear lag effect in the tensile flange. The current practice for consideration of the shear lag effect in non-rectangular walls is by means of the effective width of the flange: the method specifies an effective flange width and assumes that only the longitudinal reinforcing bars within the width contribute to resisting the tension force in the flange. ACI 318-11 (ACI, 2011) and Eurocode 8 (EC8) (CEN, 2003) specify the effective width to be taken as one quarter of the wall height. However, several investigations have indicated that the influence of the shear lag effect is aggravated by a reduction in wall length (Hassan and El-Tawil, 2003; Pantazopoulou and Moehle, 1990). The influence of the shear lag effect was thus investigated for the specimens tested in this work, which had lengths significantly shorter than their widths.

Table 4 compares the tension force in the flange calculated based on the experimental measured vertical strain with the tension force calculated based on the plane section assumption. The tensile force ratios are given for the two specimens corresponding to different levels of maximum tensile strain in the flange. The drift ratios at which these maximum tensile strains were measured are also listed in the table. As shown in the table, the experimentally derived tensile force was significantly less than the predicted tensile force. For SLW, the ratio was in the range 0.73–0.80 whereas the ratio was in the range 0.76–0.99 for STW. For SLW with maximum strains of 500–3000 $\mu\epsilon$, the ratio remained relatively unchanged and the experimentally measured tensile force was still 20% less than the anticipated force at the maximum tensile strain of 4000 $\mu\epsilon$. For STW, the ratio was found to significantly increase for maximum strains larger than 1500 $\mu\epsilon$, and the experimentally derived tensile force approached the calculated value when the maximum strain was 4000 $\mu\epsilon$.

Table 2. First yield, peak and failure points of the specimens as measured experimentally

	SLW		STW	
	Flange in compression	Flange in tension	Flange in compression	Flange in tension
First-yield displacement: mm	6.7	8.1	8.6	9.1
Lateral force at first yield: kN	61	94	83	102
Secant stiffness at first yield: kN/mm	9.1	11.6	9.7	11.2
Peak strength: kN	176	163	113	162
Lateral displacement at peak strength: mm	60.5	49.8	60.3	40
Displacement at failure: mm	—	59	—	50

Figure 9. Components of top deformation for the tested specimens

Figure 10. Vertical strains in the reinforcing bars in the flange of the tested specimens

Table 3. Comparison of experimentally derived stiffness with design methods

	SLW		STW	
	Flange in compression	Flange in tension	Flange in compression	Flange in tension
Effective moment of inertia, $I_w = K_f h_w^3 / (3E_c)$: m ⁴	0.0019	0.0022	0.0018	0.0023
Gross moment of inertia, I_g : m ⁴	0.0049	0.0049	0.0049	0.0049
I_w/I_g	0.39	0.44	0.36	0.46
Estimate of I_w/I_g (Paulay and Priestley, 1992)				
$I_e/I_g = 100/f_y + P_u/(f_{cA_g})$				
$F = 30I_e/(h_w^2 b_{wl-w})$	0.23	0.23	0.23	0.23
$I_w = I_e/(1.2 + F)$				
Estimate of I_w/I_g (Zhang and Li, 2016a)				
Flange in compression:				
$I_e/I_g = 125/f_y + 1.75P_u/(f_{cA_g})$				
$I_w = I_e/[1 + 30I_e/(h_w^2 b_{wl-w})]$	—	—	0.36	0.51
Flange in tension:				
$I_e/I_g = 185/f_y + 2.35P_u/(f_{cA_g})$				
$I_w = I_e/[1 + 40I_e/(h_w^2 b_{wl-w})]$				
First-yield displacement, Δ_y : mm	6.7	8.1	8.6	9.1
Estimate of the first-yield displacement, Δ_y : mm (Priestley and Kowalsky, 1998)				
Flange in tension:				
$\phi_y = 1.4\epsilon_{sy}/I_w$				
Flange in compression:	8.5	10.9	8.5	10.9
$\phi_y = 1.8\epsilon_{sy}/I_w$				
$\Delta_y = \phi_{yh-w}^2/3$				

Note: K_f is the secant stiffness of the wall at first yield, f_y is the yield strength of the vertical reinforcing bars, ϵ_{sy} is the yielding strain of the vertical reinforcing bars and ϕ_y is the yield curvature of the wall

Based on the effective flange width method, the flange of STW was fully effective while the effective width of the overhanging flange of SLW should be 0.635 m, which is 79% of the width of the flange. The reinforcing bars within the effective flange width, on the other hand, accounted for only 32% of the total reinforcement in the overhanging flange. It is therefore clear that the effective width method overestimated the influence of the shear lag effect in SLW while underestimating its influence in STW.

Shear strength

Both the tested specimens failed due to web crushing, which is a shear-controlled failure mechanism. This was rather unexpected considering that the shear span ratio of the specimens was 4.22, which corresponds to a slender wall (Li *et al.*, 2015; Zygouris *et al.*, 2013). The contribution of the large flange of the specimens to the flexural strength could be responsible for the failure. On the other hand, Barda *et al.* (1977), Wood (1990) and Gulec and Whittaker (2011) pointed out that the flange of the wall could also significantly affect the shear strength of non-rectangular RC walls. It is therefore believed that the existing methods require evaluation for walls of length significantly shorter than their width, and these evaluations were performed with the experimental data obtained in this work. Three methods were adopted for comparisons with the experimental data – the procedures provided in chapter 21 of

ACI 318-11 (ACI, 2011), the method proposed in EC 8 (CEN, 2003) for diagonal compression shear strength and the method proposed by Paulay *et al.* (1982).

ACI 318-11 (ACI, 2011) specifies that the shear strength is to be calculated as the combination of the shear strength contributions from the concrete and the shear reinforcement

$$1. \quad V_n = V_c + V_s$$

in which V_c is the shear strength contributed by the concrete, which is calculated as the smaller value predicted by two equations

$$2a. \quad V_c = 0.274\sqrt{f'_c}td + \frac{P_u d}{4l_w}$$

$$2b. \quad V_c = \left[0.05\sqrt{f'_c} + l_w \left(0.104\sqrt{f'_c} + \frac{0.2N_u}{l_w t} \right) \right] / \left(\frac{M_u}{V_u} - \frac{l_w}{2} \right) td$$

and V_s is the shear resisted by the stirrups, given by

$$3. \quad V_s = \frac{A_v f_y d}{s}$$

Figure 11. Comparisons of pushover curves of the tested specimens with their pushover curves from the strong axis

In addition, ACI 318-11 (ACI, 2011) specifies an upper limit of the shear strength V_n for crack control as

$$4. \quad V_n = 0.83\sqrt{f'_c}td$$

ACI 318-11 (ACI, 2011) also limits the shear in structural walls due to lateral loads in an earthquake-resistant structure to

$$5. \quad V_n \leq A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_t f_y\right)$$

in which α_c is 0.25 for $h_w/l_w \leq 1.5$, 0.17 for $h_w/l_w \geq 2.0$ and varies linearly between 0.25 and 0.17 for h_w/l_w between 1.5 and 2.0. The calculated results based on Equations 1–5 are summarised in Table 5.

Similar to the specified requirements for earthquake-resistant design in ACI 318-11 (ACI, 2011), EC8 (CEN, 2003) imposes further restrictions on the shear force in walls for earthquake-resistant design. Among them, the most significant restriction

for the tested specimens is clause 5.5.3.4.2b of EC8 (CEN, 2003), which limits the diagonal compression shear capacity in the critical regions for ductile walls to be 40% of the value outside the critical region. Therefore, the diagonal compression shear capacities for these specimens would be calculated as

$$6. \quad V_{DC} \leq 40\% \cdot v_1 t \cdot 0.8 l_w \cdot f'_c / (\cot \theta + \tan \theta)$$

in which v_1 is 0.6 for $f'_c < 60$ MPa and θ is the strut inclination in the truss model. A θ of 30° was assumed for the two tested specimens.

In fact, the equations specified by ACI 318-11 and EC 8 ignore the influence of ductility in the shear strength of the walls. Paulay *et al.* (1982) proposed an equation that relates the diagonal compression shear capacity of the wall with the ductility of the wall

$$7. \quad V_i \leq \left(\frac{0.22\Phi_{o,w}}{\mu_\Delta} + 0.03 \right) f'_c \leq 0.16 f'_c \leq 6 \text{ MPa}$$

Table 4. Ratio of experimentally derived tensile force at the flange of the base of the wall to the tensile force calculated based on sectional analyses

Maximum tensile strain in flange: $\mu\epsilon$	SLW		STW	
	Corresponding drift ratio: %	Tensile force ratio	Corresponding drift ratio: %	Tensile force ratio
500	0.23	0.73	0.10	0.76
1000	0.35	0.73	0.28	0.71
1500	0.37	0.71	0.23	0.66
2000	0.54	0.74	0.34	0.82
3000	0.72	0.73	1.05	0.95
4000	1.11	0.80	1.62	0.99

Table 5. Comparisons of experimentally derived shear strength with calculated results

	SLW		STW	
	Flange in compression	Flange in tension	Flange in compression	Flange in tension
Experimentally derived peak strength: kN	176	163	113	162
V_s based on Equation 3: kN	125	125	125	125
V_{c1} based on Equation 2a: kN	192	192	192	192
V_{c2} based on Equation 2b: kN	48	48	48	48
V_n based on Equation 5: kN	657	657	622	622
V_n based on Equation 1: kN	172	172	172	172
V_{DC} based on Equation 6: kN	173	173	173	173
V_i based on Equation 7: kN	195	195	195	195

in which V_i is the nominal shear strength calculated as $V_i = V/(t/l_w)$, $\Phi_{o,w}$ is the overstrength factor and μ_Δ is the displacement ductility of the wall. The calculated results are also listed in Table 5, based on the assumed μ_Δ of 3.

From the table, it can be seen that the lateral strength of the wall in the flange in the tension direction, which is controlled by the web crushing failure, was slightly overestimated by Equation 1 from ACI 318-11 (ACI, 2011), Equation 6 from EC 8 (CEN, 2003) and Equation 7 proposed by Paulay *et al.* (1982). Among these equations, Equations 6 and 7 were specifically proposed for the diagonal crushing. The web crushing strength was around 160 kN for both specimens, and the closest estimation was obtained from Equation 6, which specified the strength to be 173 kN. The large flange of the specimens did not seem to contribute to the web crushing strength of the wall when the flange was in tension. On the other hand, for SLW, for the flange in the compression loading direction, a lateral force larger than the predicted shear strengths of Equations 1 and 6 was observed, which was the maximum lateral strength of 176 kN. The specimen did not show indications of possible shear failure in this direction. For STW in the flange in the compression direction, the lateral strength was controlled by the flexural strength, and the shear strength was not tested; it showed a maximum lateral strength of 113 kN. More experimental data are needed for a solid conclusion to be reached, and further investigations may be required for the influences of large flange segments on the shear strength of non-rectangular RC walls.

Conclusions

This paper has reported experimental results for two non-rectangular RC wall specimens. The particular aspect of the two specimens was that the length of the wall was significantly shorter than its width. These specimens were designed in order to investigate the seismic behaviour of non-rectangular RC walls in the weak axis, which has been insufficiently researched. The following conclusions can be drawn from the experimental data and analysis.

- The two specimens loaded in the weak axis exhibited significant lateral strength, with a maximum lateral strength of 173 kN for the L-shaped specimen (SLW) and 162 kN for the T-shaped specimen (STW). The specimens were also found to be able to sustain some lateral displacements. SLW was able to sustain a lateral drift ratio of 2% with losing lateral strength, while STW was able to sustain a lateral drift ratio of 1.5%.
- Existing equations proposed for estimating the stiffness of non-rectangular RC walls were found to possess an acceptable level of accuracy for the two specimens tested. These methods include the equations proposed by Priestley and Kowalsky (1998) for estimating the first-yield displacement and the equations proposed by Zhang and Li (2016a) for estimating the effective stiffness.
- The flexural stiffness of the specimens was significantly boosted by the large flange. Due to their large flexural stiffness, both specimens failed due to web crushing, which

is a shear-controlled failure mechanism, despite the specimens having a very large shear span ratio of 4.23.

- (d) Despite the reduced length of the wall, the influences of the shear lag effect of the tested specimens were found to be very similar to walls with a 50% longer length. This indicates that the current practice of neglecting the influences of wall length on the shear lag effect may be reasonable. On the other hand, the effective flange width method recommended by the current design codes was found to be inaccurate for the tested specimens.
- (e) The shear strengths controlled by web crushing of the two specimens were found to be slightly less than predicted by current design codes. The large flange was not found to contribute significantly to the shear strength of the specimens. This is probably due to the flange being in tension. For the flange in the compression direction, on the other hand, the maximum lateral strength of the wall surpassed the shear strength estimated by ACI 318-11 (ACI, 2011) and EC8 (CEN, 2003). Further investigations may be needed to investigate the influences of large flange segments on the shear strength of the web.

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