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The Effect of the Amount, Distribution and End Anchorage Conditions of Bond Beam Reinforcement on the Behaviour of Concrete Masonry Shear Walls

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2 Reinforcement on the Behaviour of Concrete Masonry Shear Walls
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Abstract

The behaviour of square shape partially grouted concrete masonry shear walls with fixed end boundary conditions subjected to constant average axial stress and cyclic lateral loading was investigated. Two replicas were tested of each combination of horizontal reinforcement ratio (0.12%, 0.06% and 0.03%) and four end anchorage conditions (straight, 90°, 180° and shear studs). The behaviour of the walls was compared in terms of strength, ductility, stiffness and dissipated energy. While the effect of horizontal bar size was greater than that of end anchorage, neither improved end anchorage nor the smaller bar size increased yielding at critical locations on the rebars. The highest course of the wall should be reinforced and grouted as it improves performance of the walls. In addition, the amount of horizontal reinforcement did not significantly affect the strength of the shear walls, therefore changes to the shear design equations in the Canadian Standard are recommended.

Keywords: Partially grouted concrete masonry shear wall, diagonal shear failure, strength, bond beams, horizontal reinforcement, end anchorage conditions, cyclic in-plane loading, fixed end boundary conditions, shear wall design equation

1. Introduction

In modern masonry buildings, the lateral load resisting system is usually comprised of reinforced masonry shear walls. To increase the in-plane shear capacity, horizontal reinforcement is placed in such walls, either in grouted courses known as bond beams, or in bed joints. In the case of bond beam reinforcement, CSA.S304-14 (2014) specifies that the reinforcing bars should be anchored by either 180° or 90° hooks at their ends. It is assumed that if the horizontal reinforcement is not anchored properly, the loss of bond between the steel and the grout will not allow full development of its tensile capacity. On the other hand, providing hooks at the ends of reinforcing bars significantly complicates the construction. Although many studies have been conducted to investigate the influence of various parameters on the strength and behaviour of partially grouted masonry shear walls (e.g.: Fattal 1993; Schultz 1996; Voon & Ingham 2006; Minaie et al. 2010; Dhanasekar and Haider 2011; Nolph & ElGawady 2012; El-Dakhakhni et al. 2013 and Bolhassani et al. 2016), only a few have investigated the effects of end anchorage conditions (e.g.: Hoque 2013; Rizaee 2015 and Seif ElDin and Galal 2018). Hoque (2013) demonstrated that different types of end anchorage of the reinforcement in bond beams did not significantly alter the performance of walls failing in diagonal shear when subject to in-plane quasi-static cyclic loading. She surmised that the horizontal reinforcement had not reached yield, thereby removing any influence that anchorage type would have on the strength and behaviour of the walls. More recently Seif ElDin and Galal (2018) examined the effects of three horizontal reinforcement end anchorage conditions (straight, 90° hook and 180° hook) in fully grouted masonry shear walls. Although they concluded that the 180° hook was the most efficient, the strength and ductility were only slightly

greater (5% and 15%, respectively) than when straight bars were used. Considering they tested only one specimen of each type, these values are within the normal variation of masonry. Seif Eldin and Galal (2018) suggested that to facilitate construction straight bars could be permitted in fully grouted moderately ductile squat shear walls. However, to apply this recommendation to partially grouted shear walls requires verification, as partially grouted walls can behave differently to fully grouted walls. Thus, the focus of this study was to investigate the influence of the end anchorage conditions of the horizontal reinforcement and subsequently its size (amount) and distribution, on the behaviour of partially grouted squat masonry shear walls that fail in diagonal shear. Walls with two sizes of horizontal reinforcement and four end anchorage conditions were tested. Strain gauges were placed on the horizontal reinforcement at the locations where cracking (and therefore maximum tensile stresses) was expected, in order to investigate the strain and yielding of the horizontal bars throughout the test.

The second focus of the study was to determine the effect of the amount and distribution of shear reinforcement on the in-plane lateral behaviour of partially grouted masonry shear walls. Most researchers agree that providing shear reinforcement improves the general performance of masonry shear walls and crack distribution, e.g. (Tomaževič and Zarnic 1985; Shing et al. 1989; Voon and Ingham 2006; Haach et al. 2007 and Oan and Shrive 2010). However, while some concluded that shear reinforcement can improve the maximum in-plane shear capacity (Priestley and Bridgeman 1974 and Khattab and Drysdale 1993), others have concluded that providing horizontal shear reinforcement has a negligible effect on the shear strength (Nolph and ElGawady 2012; Hidalgo-Leiva et al. 2016; Oan and Shrive 2015 and Rizaee and Shrive 2018) or its effect

on shear strength is not completely clear (Shing et al. 1989; Foltz and Yancey 1993; Schultz et al. 1998 and Haach et al. 2010). The results of the experiments conducted by Rizaee (2015) were therefore analyzed carefully to investigate the effects of end-anchorage, and amount and distribution of horizontal reinforcement on the shear strength of partially grouted squat masonry shear walls failing in diagonal shear.

2. Experimental Program

Three shear failure modes (sliding, diagonal and flexural failure), or combinations of them are possible when a shear wall is subject to lateral load. Usually, when a wall is subjected to high lateral and low axial loads the lateral load easily exceeds the friction resistance in the bed joints and the dowel action of vertical reinforcement and results in sliding failure. Diagonal shear failure is likely to happen when moderate axial load is applied on walls with low aspect ratio (height to length ratio). Having reinforcement in both the horizontal and vertical directions delays failure by distributing the shear stress to the reinforcement and cracks over the wall and results in more ductile failure. The most ductile failure of all is flexural shear failure. This failure occurs mostly in slender walls (aspect ratio > 1.5) in which yielding of vertical tensile reinforcement and horizontal cracks at the heel region occur together with possible buckling of vertical reinforcement and crushing of masonry units in the toe region. In the experiments analyzed here, diagonal shear failure was targeted by having walls with an aspect ratio of one, applying a moderate amount of axial load and providing reinforcement in both the horizontal and vertical directions. The predicted strengths and type of failure for these walls were determined according to the CSA.S304-04 (2004) and multiplied by the average factor of safety found from a previous

experimental study (Dickie and Lissel 2010) resulting in the prediction of diagonal shear failure at a load of 320 kN.

2.1 Wall Specimens

Seven groups of two shear walls each were constructed. For proper statistical analysis, larger sample sizes are necessary but are difficult in structural testing due to the time and expense related in carrying out such tests. However, having two specimens provides more confidence in the result. The different groups of walls and their relevant variables are shown in Table 1. The variable parameters were the horizontal placement, size and end anchorage condition of rebars. Two sizes of horizontal bar, 15M and 10M, with diameters of 16.0 and 11.3 mm, respectively, were used. The reinforcement was placed in the bond beams located in the middle and top (5th and 9th), middle (3rd and 6th) or singly in the middle (5th) courses of the walls (in this latter group, the 9th course was just grouted). The 90° and 180° hook specifications, such as the bend diameter and the hook length, are governed by clause 12.4.5 of the Design of Masonry Structures, CSA.S304-14 (2014). Stud-ended bars were prepared according to clause 7.1.4 of the Design of Concrete Structures, CSA.A23.3-14 (2014), which requires a circular discs of 5 mm thickness with an area of at least 10 times the rebar size; therefore, 50 mm diameter discs were used with 15 mm diameter holes in the centre to facilitate welding of the rebar to the shear stud. A few pictures of the construction of the walls, end anchorage conditions and placement of rebars in bond beams are shown in Figure 1. As the desired type of failure was diagonal shear, the walls were constructed with an aspect ratio of one, with dimensions of 1790 × 1790 mm (length × height), using 390 × 190 × 190 mm (length × height × thickness) hollow concrete blocks. As

shown in Figure 1, vertical reinforcement of 15M diameter was placed at 800 mm spacing in the middle and in each end core of each wall (vertical reinforcement ratio of 0.18%), in accordance with common practice for masonry wall construction in North America (Dickie and Lissel 2011). Dhanasekar and Haider (2004) stated that uniform spacing of vertical reinforcement leads to more gradual degradation in the lateral strength of a shear wall and higher ductility. into simulate typical construction practice and to create realistic boundary conditions, in addition to preventing flexural failure or overturning of the specimens and to connect the vertical reinforcement with the base beam, 600 mm long 15M dowel bars were welded to the base beam where the vertical reinforcement was placed. The vertical reinforcement was then spliced to the dowels using wires.

2.2 Test Setup and Instrumentation

The locations of the reinforcement and strain gauges for the various groups of walls are shown in Figure 2. As the cyclic shear lateral load applied at the top is transferred diagonally to the toes of the wall (Dhanasekar and Haider 2004), the highest tension stresses are expected to occur in the middle and the four corners of the walls. The results of the tests on group A and B walls were studied prior to the construction of the remaining groups. The bar diameter in these groups was reduced in order to increase the chances of observing the influence of anchorage on bar yielding and subsequently on the strength and behaviour of the walls.

2.3 Material Properties

The average and standard deviation of the compressive strengths of the materials used in each group of walls are presented in Table 2. For each property five to eight samples were tested on the same day as the walls. Tests were conducted according to the requirements of CSA.S304-14

(2014), Design of Masonry Structures and CSA.A179-14 (2014), Mortar and grout for unit masonry. In addition, three samples of each of the 15M and 10M rebars were tested for tensile failure resulting in yield strains of $2380 \pm 104 \mu\epsilon$ (mean $\pm$ SD) and $2625 \pm 83 \mu\epsilon$ and yield stresses of 448 ± 1.8 and 456 ± 12.7 MPa respectively. Prebagged, preblended mortar and grout from the same manufacturer were used to minimize variations and inconsistency. However, two different dry mix type S mortars were used; for group A and B, High Strength Lime Based mortar, and for the remaining walls Portland Lime and Sand mortar, providing half the strength of mortar used in the first two groups.

2.4 Test Arrangement

The test arrangement, shown schematically in Figure 3, was based on that developed by the ESECMaSE research consortium (Fehling and Schermer 2009), being modified for reinforced masonry walls (Dickie and Lissel 2010). The arrangement is meant to simulate the boundary conditions of a shear wall in a middle story of a building subjected to lateral loading. The walls were constructed on steel, C380 $\times$ 63.1, channel base beams on which wave pattern weld beads were welded to increase the sliding shear resistance between the base beam and the wall. The base beams were bolted at four locations on each side to the load floor to prevent any overturning and sliding between the base beam and the floor. Transducer 9 was recording slippage between the capping beam and the top of the wall, transducer 10 was recording lateral displacement (slip) between the base beam and the laboratory floor. Simultaneously, and transducers 7 and 8 were recording vertical displacement between the wall and the base beam. Since none of these transducers recorded any movement, it was confirmed that this boundary condition prevents any

movements at the base of the wall and satisfies fixed end boundary conditions. The vertical dowels welded to the base beams prevented sliding between the wall and the base beam. A W450×140 steel beam with large longitudinal stiffness was used as the capping beam. This size of steel beam was selected in order to resemble the effect of a floor slab. The bottom flange of the capping beam was mortared to the top of the wall one day before the test, using fast setting mortar. A wave pattern weld bead was welded to the bottom flange to increase engagement with the mortar and better transfer the horizontal force to the wall. There were no other dowels or mechanical connections between the wall and the capping beam, to represent a realistic wall to floor connection. Displacements at various locations on the wall were measured using Linear Strain Convertors (LSCs). Their placement on the wall is shown in Figure 3. LSC 1 was used to control the application of the horizontal displacement according to the protocol.

A hydraulic actuator applied the horizontal load at the midpoint of the capping beam to minimize the inconsistency between the positive (push) and negative (pull) loading directions. The loading protocol was developed based on Fehling and Schermer (2009) protocol. According to that displacement and load at crack and displacement at maximum load were needed to be determined. In this experiment the values estimated from the preliminary tests conducted by Dickie and Lissel (2010) were used. In order to record the post-peak behaviour of each specimen, horizontal displacements were applied in three phases: up to a maximum lateral displacement of 1 mm, before estimated crack, in a cycle, the displacement was applied at 0.1 mm/s, with maximum displacement being increased 0.25 mm per cycle; between 1 and 15 mm maximum displacement per cycle, post crack, the displacement rate was increased to 0.25 mm/s and the increments in peak

displacement were 0.5 mm; above 15 mm, post peak displacement, the increments in maximum displacement in a cycle were 1 mm with the displacement rate of 0.5 mm/s. Two cycles of each displacement amplitude were applied, each held for two seconds. The tests were terminated when either a significant drop (about 20% of peak capacity) had occurred in either direction, or the walls were severely cracked, defining the state of failure.

Force controlled axial load was applied through two actuators located 500 mm from the centre of the wall. An initial moderate axial load of 409 kN was applied to all the test specimens. This load was calculated as a realistic floor load on a load bearing mid level story wall by Dickie and Lissel (2011), inducing a stress of approximately $0.2f'_m$, or about 2 MPa over the net area of the wall. In the Dickie and Lissel (2011) study the average f'_m was 10 MPa and the net area of the specimen was 204600 mm². The total axial load was expected to remain constant throughout each test. As the applied horizontal displacement and the corresponding load changed magnitude and direction, the vertical load was adjusted so that the total moment remained zero at the mid height of the wall, as shown in Figure 4 (b). This loading scheme was used in wall groups A and B. However, a problem arose when the applied horizontal load exceeded 227.2 kN. At this point for a push load, the vertical load adjustment required that the actuator closer to the horizontal actuator (N1 in Figure 4 (a)) exert more than the original total compressive load and the far actuator (N2) exert tensile load in order to maintain zero moment at the mid height of the wall. However, the vertical actuators were not capable of applying tensile forces, so while the closer actuator applied more than 409 kN the far actuator applied no load. Since the actuators could not be moved farther apart, a solution to this situation was to change the location of zero moment from the mid-height of the

wall. Considering the maximum shear strength of the walls in groups A and B the critical horizontal load, after which the vertical actuators need to go into tension, was increased to 320 kN. This resulted in changing the location of zero moment to 640 mm below the level of horizontal load application (415 mm below the top of the wall) and in unequal moments at the top and bottom of the wall. The rest of the tests were conducted using the adjusted loading method, shown in Figure 4 (c).

3. Data Analysis and Test Results

3.1 Bilinear Idealization and Displacement Ductility for Data Analysis

To resist seismic loads safely, masonry shear walls should be able to undergo large deformations while remaining stable. The ability to do so is assessed through displacement ductility, the ratio of the lateral displacement of a wall at a certain level of strength to the lateral displacement at cracking, derived using bilinear idealization. Different approaches for bilinear idealization and consequently displacement ductility have been suggested by Tomažević and Zarnic (1985), Shing et al. (1989) and Minaie et al. (2010). Bilinear idealization presents an idealized elasto-plastic load-displacement curve. In this study an idealized bilinear load-displacement curve was determined through an approach similar to the one suggested by Tomažević and Zarnic (1985), in which the bilinear idealized curve has the same amount of energy (the area under the curve) as the experimental envelope. This approach is illustrated in Figure 5.

According to Tomažević and Zarnic (1985) the displacement and load at which the first diagonal crack occurs define d_{cr} and L_{cr} . Here d_e was used instead of d_{cr} . d_e is where there is a noticeable reduction in the elastic stiffness (k_e) in the hysteresis envelope. The ultimate displacement, d_u , is

the point where the idealized curve intersects the experimental envelope. However, in this study, since the intersection point for some walls occurred very early after yielding, the point where the strength had reduced 20% from the maximum was chosen to represent d_u . In some cases, the failure load was greater than 80% of the maximum strength of the wall, so the maximum displacement was considered to be the ultimate displacement that the test was conducted to even though the load had not degraded to lower than 80% of the maximum load. To calculate H_u (the idealized shear strength of the wall) and subsequently the displacement ductility (d_u/d_e), equations below were used.

$$\begin{aligned} a) \ k_e &= \frac{H_{cr}}{d_{cr}} & b) \ d_e &= \frac{H_u}{k_e} & c) \ A_{env} &= H_u d_{max} - \frac{H_u d_e}{2} & (1) \\ d) \ A_{env} &= H_u d_{max} - \frac{H_u^2}{2k_e} \end{aligned}$$

- A_{env} , is the area under the experimental envelope, calculated from the actual hysteresis envelope.
- H_u , is the ultimate strength of the wall based on the idealized curve, not based on design.
- d_e , is the elastic displacement of the wall, which is the point where the stiffness of the idealised wall becomes zero.
- d_{max} , is the maximum displacement of the wall.
- d_{cr} , is the displacement where the initial stiffness of the actual wall changes. (Representing the occurrence of significant cracks).
- H_{cr} , lateral load at d_{cr} .

3.2 Test Results

The test results and calculated values are summarized in Table 3 and Table 4 for each of the walls at critical stages of the test in both directions. Although the first visible cracks on the walls were marked and recorded, the stiffness of the walls did not necessarily change at that point. Therefore, the cracking displacement was defined as the first significant crack at which the stiffness of the wall, the slope of the load-displacement envelope, decreased, from which one displacement ductility value for each wall is calculated. In order to compare the behaviour of the different groups of walls, their shear strengths should be normalized. The most common method, which is also applied in this study, is to divide shear strength by the square root of their respective masonry compressive strengths (f'_{wm}) as determined from prism tests (Matsumura 1987; Shing et al. 1989 and Voon and Ingham 2006). Displacements at which yielding was recorded at the location of each strain gauge is also reported in Table 5.

4. General Observations

To facilitate comparison of the wall behaviours, lateral load-displacement graphs including hysteresis loops, envelopes and the bilinear idealized curves are presented in Figure 6 (groups A and B), Figure 7 (groups C, D and E) and Figure 8 (groups F and G). The crack pattern at failure and the locations of horizontal reinforcement strain gauges are presented in Appendix A, Fig. A1, Fig. A2 and Fig. A3 respectively.

The walls in this experiment resisted drifts in the range of 0.31-0.56%, Table 3 and Table 4. In comparison to the drift values of about 1% reported by El-Dakhkhni et al. (2013) and Seif Eldin and Galal (2018), the values here are low. This is possibly due to the axial stress here being twice

what applied in the other two studies, the boundary conditions being fixed-fixed as opposed to cantilever and/or the differences in reinforcement ratio and grouting. The walls in this experiment were partially grouted with a vertical reinforcement ratio of 0.18% while the walls in the other two experiments were fully grouted with vertical reinforcement ratios of 0.79% to 1.32%, more than 4 times the ratio in this experiment. In addition, the horizontal reinforcement and spacing in the tests here gave horizontal reinforcement ratios of 0.03, 0.06 and 0.12% while the specimens in their tests had horizontal reinforcement ratios of 0.070 to 0.13%.

The displacement ductility values range from 2.6 to 6.2 with a median and average value of 4.6 and 4.7 respectively, while the ductility values reported by Seif ElDin and Galal (2018) ranged between 2.3 and 4.2 with median and average values of 3.7. In addition, although the level of axial stress was 10% to 30% greater in groups A and B than in subsequent tests, on average this did not affect the ultimate displacement significantly. Shedid et al. (2008) also found that displacement at ultimate is not very sensitive to the distribution or amount of vertical reinforcement or amount of axial load.

4.1 Discussion of Results for Groups A and B

The overall behaviour of the walls in groups A and B is shown in Figure 6. At the beginning of the test on wall 1, one vertical actuator went into resonance, so the test was stopped. Cracks present from this first test could have initiated the subsequent catastrophic and unsymmetrical failure in the second testing of this wall, at the lowest displacement of all walls. In wall 1, no cracks were observed until failure occurred in course 9 while many cracks were created around

275 course 5. The other three walls exhibited ductile behaviour and X pattern cracking with more
276 crack formation in the middle part of the walls. Although the walls in group B had straight
277 horizontal rebars they reached the highest ultimate displacement of 9.5-10.0 mm. Walls in
278 groups A and B experienced higher axial loads compared to the rest of the walls resulting in
279 higher shear strengths.

280 Looking at Table 5, in all walls in groups A and B, the rebar in course 5 yielded at the mid-
281 length (SG4) and 200 mm to the left (SG3) and right (SG5). Yielding at these three locations in
282 group A walls, 180° hooks, occurred when or after the maximum strengths were reached,
283 showing no contribution of yielding to the strength of the walls. In group B walls, without end
284 anchorage, yielding occurred before maximum strengths were reached implying a possible
285 contribution of yielding to the strength. According to CSA.S304-14 (2014), clause 12.4.2.4, the
286 required development length of a 15M rebar is 470 mm. The minimum available development
287 lengths at the locations of SG4 and SG3 (SG5) were 850 mm and 650 mm, respectively, when
288 straight rebar is considered. Therefore, there is sufficient length to develop yield at the locations
289 of SG4, SG3 (SG5) regardless of the end anchorage conditions. Although near yielding strains
290 were recorded in the other two gauges (SG2 and SG6), they only occurred after maximum
291 strength was reached. Although there is not enough length theoretically to develop yield at the
292 locations of SG7 and SG1 near the ends of the rebar, yield was recorded at SG7 in group A and
293 at SG1 in group B walls close to the end of the tests. Thus, having 180 ° hooks and subsequently
294 longer development length in the horizontal rebar in group A did not increase the possibility of
295 yielding at the ends of the rebar. Comparing the strength of the two groups as well as yielding in

the rebar shows that the end anchorage conditions did not affect either of these quantities in this experiment.

4.2 Comparison of Results for in other Wall Groups

In groups C, D and E the 10M rebars were placed in the 5th and 9th courses with 180° hooked ends, 90° hooked ends and shear studs, respectively. In groups F and G, 90° hooked ends were used with the reinforcing bars placed in the 3rd and 6th courses in group F and the 5th course only in group G. The overall behaviour of each wall in this set of tests is shown in Figure 7 and Figure 8.

Table 5 shows that yielding occurred at or after maximum strength was reached for all strain gauges which recorded yielding. This demonstrates that yielding of horizontal reinforcement at the locations where end anchorage conditions seem to have an influence did not affect the strength of the walls. Similar to groups A and B, a few cracks were formed at the 9th course in these groups of walls but the rebar in this course was never subject to high strain. Therefore, the contribution of a reinforced bond beam in the top course of a wall to the shear strength of that wall is questionable. The rebar in course 9 mostly displaces as a rigid body back and forth as the wall undergoes cyclic displacement. Cracks develop mostly at the two ends of this course during the final stages of the test, well after the peak load has been passed. With major cracks near the ends of the course, there is no development length for the rebar in the small sections between the cracks and the ends of the wall. While X-pattern cracking was also observed in these groups, walls 6, 7, 10 and 11 exhibited more distributed crack patterns. The cracks were mostly concentrated through the three middle courses, decreasing towards the top and bottom and showing the least number of cracks in the 9th

course. At the two bottom corners of the walls, cracks were more concentrated in the toe of the wall in the pull direction with toe crushing in some cases. The walls usually exhibited ductile behaviour and gradual strength reduction, especially in the push direction (left to right in Figure 3) with push being the direction of first applied displacement.

5. Comparison of Results

5.1 Strength

In general, walls in groups A and B demonstrated higher strengths in both directions than the rest of the walls. The higher strength did not appear to be caused by the higher horizontal reinforcement ratio (0.12% compared to 0.06% and 0.03% in other groups) but more probably by the walls being subjected to axial stresses higher than 2 MPa, rising to 2.2 - 2.6 MPa in the final stages of the test. As explained above, the point of zero moment in the walls and subsequent vertical load application was adjusted to prevent the axial stress from increasing beyond 2 MPa in subsequent tests.

The normalized load-displacement envelope curves for all the walls are shown in Figure 9. While W1 to W4 (groups A and B) show higher maximum strengths, possibly due to the higher axial stresses or normalization with lower prism strengths, the rest of the walls show similar load-displacement curves with the typical variability of masonry. Thus, encouraging that the end anchorage conditions and the amount of horizontal reinforcement did not affect the shear strength of these walls.

Oan and Shrive (2010; 2015) concluded that in reinforced masonry shear walls only the shear strength of masonry and the vertical reinforcement contribute to wall lateral strength and that horizontal reinforcement starts to contribute to the shear resistance after cracking of masonry and by improving the post cracking performance of the wall. Hidalgo-Leiva et al. (2016) also showed that reinforced bond beams did not improve the maximum lateral in-plane strength of the walls. In addition, Rizaee and Shrive (2018) conducted ANOVA test on the results of their experiments and showed that horizontal reinforcement did not change the strength of masonry shear walls in a statistically significant way (significance taken as $p < 0.05$). Similar to the above studies, the results here demonstrate that the horizontal reinforcement ratio does not change the maximum lateral strength of concrete masonry shear walls.

The shear strength values for the test walls were also calculated according to CSA.S304-14 (2014) with no material safety factors and using the weighted prism strengths. In all the cases the shear strength maximum limit governed. However, the actual shear strengths were on average 17% greater than the calculated strength while two walls (W6 and W10) showed higher calculated strength than actual and three walls were less than 10% different in strength. These differences show that the equations in the standard are quite accurate for the walls tested.

5.2 Recorded Strain and Yielding of the Reinforcement

Strain and residual strain increase in the reinforcing bars as the cracks develop and displacement increases. This action maintains the integrity of the walls, controls crack width and produces ductile behaviour. Before the first cracks are formed on the 5th course, the tensile strain in the

355 rebar is around $200 \mu\epsilon$: this is in agreement with Oan and Shrive (2015) and as with their results,
356 as soon as the cracks are formed, residual strains develop (1000 - $1500 \mu\epsilon$) and strain continues to
357 increase with displacement and increasing crack width.

358 Strain vs. lateral displacement curves for the strain gauges in Wall 4 are shown in Figure 10 as
359 an example. The strain-gauges did not record residual strain until the first cracks formed (in 1.5
360 mm cycle). SG4 reaches strains up to $19000 \mu\epsilon$ and both SG3 and SG5 reach yielding in the 5.5
361 and 7 mm cycles respectively. SG1 (the gauge at the left hand end on the reinforcing bar in
362 course 5) recorded higher strains when the wall was “pushed” (shear load to the right) while SG7
363 (at the right hand end of the bar) recorded higher strains when the wall was “pulled” (shear load
364 to the left). These two strain gauges reached around 1000 - 1200 residual strain when the rebar
365 yielded. It was expected that there would be more yield of the smaller diameter horizontal
366 reinforcement. The strain gauges in the top course bond beam, SG8 (SG9) do not reach any high
367 strains and only record strains up to maximums of 400 to $1200 \mu\epsilon$.

368 Amongst the walls with $10M$ rebar, the reinforcement in course 5 did not always reach yield at
369 both ends, except in the case of the 180° hook (wall 5 in group C). In order to see the influence
370 of reducing bar size on the yielding, yielding at the location of the two end strain gauges (SG1,
371 SG2 and SG6, SG7) is compared ($15M$ vs. $10M$) in Table 6. By decreasing the horizontal rebar
372 diameter, the proportion of bars yielding increased at location 1, remained the same at location 2
373 and 7 and decreased at locations 6. Overall, contrary to expectations, reducing the bar size did

not increase the occurrence of yielding in the rebar at the bar ends. The reason could be that the yield strain for the 10M bars was 10% greater than for the 15M bars.

5.3 Displacement Ductility

The average displacement ductility values (d_u/d_e) in each group of walls in both directions are presented in Figure 11. The ductility of the walls in group B (15M - straight) was higher than group A (15M - 180° hooked) in both directions. One reason could be the test problem at the beginning of all tests happening in wall 1 in group A which resulted in the lowest ultimate displacement and lower average displacement ductility in this group. In addition, the average ductility of group D (10M - 90° hook) in both directions was higher than group C (10M - 180° hook) and group E (10M - shear studed). While the differences are small, the need for 180° hooks at the ends of a rebar in this form of construction is clearly questionable: 90° hooks appear to be quite adequate. It is not clear why the walls with 180° hook horizontal bar showed less ductility than 90° hook walls. Finally, the average ductility of the walls in group G (10M-only 5th course) is the lowest. Thus, although the 9th course only gets few cracks at the ends, having reinforcement in this course appears to improve ductility. It is recognized as good practice to have a bond beam at the top of a hollow or partially grouted concrete masonry wall, and the benefit in terms of maintaining wall integrity is seen here.

5.4 Stiffness

The lateral (shear) load is distributed between the shear walls of a building based on their stiffness: the higher the stiffness, the higher the share of the lateral and possibly torsional loads. During the

application of cyclic load, cracks propagate through a wall and as a result, the stiffness degrades. In this study, the effective stiffness of the walls which is the equivalent elastic stiffness from the bilinear idealization of load displacement curves, are compared.

The average experimental effective stiffness, actual values, and the average normalized effective stiffness for each group of walls in both directions are presented in Figure 12. In the calculation of normalized effective stiffness, the normalized shear strength is used. As illustrated in Figure 12, the effective stiffness in the pull is always smaller than in the push direction. The reason could be that the test always starts in the push direction which could cause some minor damage and lower the effective stiffness in the pull direction. Groups A and B walls (with 15M bars) were the stiffest, while the stiffness of the walls with 10M horizontal bars were almost equal in both directions and the experimental stiffness of group G walls (10M-90° hook in 5th course only) and normalized stiffness of group E (shear studded 10M bars in 5th and 9th course) were the lowest.

5.5 Dissipated Energy

The capacity to dissipate energy is another way to evaluate the performance of masonry shear walls. The input energy, Figure 13 (a), is the work done by the horizontal load actuator to displace the wall to the peak displacement of that cycle, which is the area under the hysteresis loop in that cycle; while, the dissipated energy is the area enclosed by the hysteresis loop, Figure 13(b). The dissipated energy in each cycle is the area enclosed by the hysteresis loop at that cycle. Mean values of dissipated energy at first cracking, maximum applied load and ultimate idealized load are presented in Figure 14. At cracking, the dissipated energy is less than 0.5

kN·m in all the groups. Dissipated energy increases as the applied displacement increases, with the highest average values in group B, being an outlier. Comparing walls in groups A and B, in contrast to expectation, the walls horizontally reinforced with straight bars (group B) not only reached higher ultimate displacements but also dissipated more energy than walls with 180° hooks at the ends of the horizontal rebars (group A).

6. Conclusions

In order to determine the influence of the end anchorage conditions of the reinforcement in bond beams on the performance of masonry shear walls, four types of end anchorage conditions were studied. In the tests described here, it was found that the end anchorage conditions had no effect on the yielding of the horizontal reinforcement at its ends nor on the shear strength of the walls.

In these tests, the performance of walls having 15M horizontal rebar with 180° hooks at each end was not significantly different to that of walls having straight horizontal bars. This finding is in agreement with that of Hoque (2013). Similar to this study, Seif ElDin and Galal (2018) studied the effects of three horizontal reinforcement end anchorage conditions (straight, 90° hook and 180° hook) in fully grouted masonry shear walls. Although they concluded that 180° hooks were the most efficient, the strength was only slightly greater (5%) than when straight rebars were used. As straight rebars or 90° hooks facilitate construction, their possible use in moderately ductile squat shear walls should be investigated.

The maximum strain recorded in the reinforcement in the top (9th) course increased when the bar size was reduced from 15M to 10M, but it never reached close to yielding. The rebar in the top

434 course moves as a rigid body as the wall moves and appears to make no contribution to the
435 ductility or the strength of the wall. However, grouting and reinforcement of this course
436 improves the ductility, displacement capacity and energy dissipation by helping maintain wall
437 integrity. Indeed, Khattab and Drysdale (1993) mentioned that the distribution of both vertical
438 and horizontal reinforcement can positively affect the ductility in a masonry shear wall.
439 Having the top (9th) course grouted but unreinforced resulted in the least amount of energy
440 dissipation and ductility, but slightly higher strength. Therefore, to maintain the performance of
441 masonry shear wall in similar situations current good practice of reinforcing the top course of a
442 wall should be continued.

443 In assessing the influence of end anchorage and distribution of 15M and consequently 10M
444 horizontal reinforcement on the performance of the walls, it was found that, in agreement with
445 the findings of Hoque (2013) for 15M reinforcement, changing the end anchorage of 10M
446 horizontal reinforcement did not affect any of the performance characteristics of the shear walls
447 significantly. The grout was carefully compacted here to engage the full length of the
448 reinforcement to ensure composite action between the masonry and the reinforcement. The
449 effects with insufficient compaction for composite action are not known.

450 Walls reinforced in the 5th and 9th courses with 15M rebars performed slightly better in terms of
451 strength, ductility and stiffness than walls reinforced with 10M rebars located in the 3rd and 6th
452 courses implying that the effect of bar size on the performance of the walls is more pronounced
453 than the type of end anchorage. The reason could be that in the former case, the horizontal
454 reinforcement ratio considering the cracked zone of the wall (lower 7 courses) is 0.075%,

whereas in the latter case it is 0.15%. This is in agreement with the suggestion of Nolph and ElGawady (2012) and Voon (2007) who stated that horizontal reinforcement above the limit of 0.1% does not improve performance of a shear wall.

Considering the difference in the applied axial loads and conducting ANOVA tests on the normalized strength of the walls, it was shown that the amount of horizontal reinforcement does not actually affect the shear strength of the walls in a statistically significant way, which is in agreement with findings of Ghanem et al. (1992); Schultz et al. (1998); Oan and Shrive (2010 and 2015); Hidalgo-Leiva et al. (2016) and Rizaee and Shrive (2018). The results of the tests here also suggest that masonry shear wall strength is independent of the horizontal reinforcement ratio, as found by Haach et al. (2010). These latter authors found that their unreinforced wall had the lowest shear strength followed by the wall reinforced only in the horizontal direction: they noted that if only shear reinforcement is considered, without the influence of vertical reinforcement or the applied axial load, it does not have significant influence on wall strength. In addition, Foltz and Yancy (1993) observed inconsistency in the effect of horizontal reinforcement ratio on the maximum shear strength of a wall. Based on these findings, it is recommended that the equations for the design of masonry shear walls should be changed in a way to reflect these findings and that horizontal reinforcement alone does not have an effect on the shear capacity of masonry shear walls.

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Table 1: Wall Specimens and the Variable Parameters

Group	Wall Number	Bond Beam Course #	Horizontal Rebar Spacing (mm)	Horizontal Reinforcement Size (Ratio)	Anchorage Type
A	1,2	5,9	800	15M* (0.12%)	180°
B	3,4				Straight
C	5,6			10M† (0.06%)	180°
D	7,8				90°
E	9,10				Shear Stud
F	11,12	3,6	600		90°
G	13,14	5,9 (unreinforced)	NA	10M (0.03%)	90°

* 15M = 16 mm diameter

† 10M = 11.3 mm diameter

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Table 3: Test Results/Calculated Values in Push Direction

Group	Wall	Crack			Maximum			Failure			Bilinear Idealized Values					
		H _{cr} (kN)	d _{cr} (mm)	H _{cr} /H _{max}	H _{max} (kN)	Normalized H _{max}	d _{@Hmax} (mm)	H _{@Dmax} (kN)	d _{max} (mm)	H _{@Dmax} /H _{max}	d _e (mm)	H _u (kN)	drift (%)	μ _d (mm/mm)	P _{@Hmax} (kN)	Wall Remark
A	1	194	1.0	0.84	232	62.1	3.0	170	5.5	0.73	1.1	218	0.31	4.9	449	15M, 180° hook
	2	227	1.0	0.70	327	87.3	3.0	236	7.5	0.72	1.2	274	0.42	6.2	499	
B	3	264	1.5	0.76	346	92.5	4.0	278	10.0	0.80	1.8	321	0.56	5.5	517	15M, Straight rebar
	4	238	1.5	0.84	282	76.8	9.0	251	9.5	0.89	1.6	259	0.53	5.8	505	
C	5	185	1.5	0.80	232	56.5	4.0	182	8.0	0.79	1.8	218	0.45	4.5	409	10M, 180° hook
	6	169	1.5	0.89	191	46.6	4.5	177	8.0	0.93	1.6	186	0.45	4.9	409	
D	7	194	1.5	0.79	244	59.5	5.0	190	8.5	0.78	1.8	228	0.47	4.8	409	10M, 90° hook
	8	184	1.5	0.83	223	54.5	4.5	200	9.0	0.90	1.8	220	0.50	5.0	409	
E	9	147	1.5	0.65	226	55.1	6.5	157	9.0	0.70	2.1	209	0.50	4.2	409	10M, Shear studs
	10	177	1.5	0.83	213	51.9	5.0	144	7.5	0.68	1.6	193	0.42	4.6	409	
F	11	172	1.5	0.81	211	42.0	5.0	167	8.0	0.79	1.7	198	0.45	4.6	409	10M, 90° hook, BB: 3 rd and 6 th
	12	152	1.5	0.73	209	41.0	4.5	152	8.5	0.73	2.0	197	0.47	4.4	409	

Table 2: Materials Compressive Strengths

Group	Masonry Block (MPa) Mean±SD	Mortar (MPa) Mean±SD	Grout (MPa) Mean±SD	Fully Grouted Prism (f'_{gm}) (MPa) Mean±SD	Hollow Prism(f'_{hm}) (MPa) Mean±SD	Weighted Prism Strength (f'_{mw}) (MPa)*	Characteristic Compressive Strength (f'_m) † (MPa)	Age (Days)
A	35.7 ±3.5	22.1 ±3.3	25.7 ±2.4	10.5 ±2.3	18.6 ±2.8	13.3W4: 12.7	10.0 W4: 9.8	30
B			W4: 21.7 ±2.3	W4: 9.5 ±2				
C	29.3 ±2.1	9.5 ±1.1	25.2 ±6.3	12.7 ±2.4	22.3 ±2.2	16.0	13.0	28
D		11.5 ±0.6	31.7 ±2.1					
E		11 ±2.4	31.8 ±3.6					
F	20.1 ±2.2	9.3 ±1	24.7 ±1.7	8.8 ±1.2	14.7 ±0.6	10.8	9.9	
G		11.6 ±2.4	29.3 ±1.8					

* $f'_{mw} = \frac{\frac{1}{3} \cdot f'_{gm} \cdot 0.9 \cdot 190 \cdot 1000 + \frac{2}{3} \cdot f'_{hm} \cdot 0.9 \cdot 75500}{\frac{1}{3} \cdot 190 \cdot 1000 + \frac{2}{3} \cdot 75500}$ in which 75500mm² is the effective mortared area for one meter of 190 mm concrete block wall and 0.9 height to thickness correction factor for prism strength is based on clause D.6.5, table D.1 in annex D of CSA-S304-14 (2014).

† f'_m = f'_{mw} - 1.64 SD, so the actual compressive strength exceeds the specified compressive strength 95% of the time according to Annex C of CSA.S304-14 (2014)

G	13	151	1.5	0.86	175	28.0	6.0	162	7.5	0.93	1.7	172	0.42	4.4	409	10M, 90° hook, 5 th course reinforced
	14	156	1.5	0.68	230	49.0	5.5	174	8.0	0.76	2.0	212	0.45	5.3	409	

H_{cr} (kN): Horizontal load when the first noticeable crack occurs

d_{cr} (mm): first noticeable crack

H_{cr} / H_{max}

H_{max} (kN): Maximum lateral load tolerated by the wall

Normalized H_{max} : Normalized maximum lateral load

$d_{@Hmax}$ (mm): Displacement at the maximum lateral load

$H_{@Dmax}$ (kN): Horizontal load at the maximum displacement

d_{max} (mm): Maximum displacement

$H_{@Dmax} / H_{max}$: Ratio of horizontal load at the maximum displacement to the maximum horizontal load

d_e (mm): Elastic displacement

H_u (kN): Ultimate horizontal load

drift (%): Ratio of displacement

μ_d (mm/mm): ductility ratio which is du/de

$P_{@Hmax}$ (kN): Axial load at maximum shear load

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Table 4: Test Results/Calculated Values in Pull Direction

Group	Wall	Crack			Maximum			Failure			Bilinear Idealized Values				
		H_{cr} (kN)	d_{cr} (mm)	H_{cr}/H_{max}	H_{max} (kN)	Normalized H_{max} (kN)	$d_{@H_{max}}$ (mm)	$H_{@D_{max}}$	d_{max} (mm)	$H_{@D_{max}}/H_{max}$	d_e (mm)	H_u (kN)	drift (%)	μ_d (mm/mm)	Wall Remark
A	1	-182	-1.0	0.67	-273	-73.0	-5.5	-273	-5.5	1.00	-1.2	-226	0.31	4.4	15M, 180° hook
	2	-184	-1.5	0.67	-276	-73.8	-7.0	-255	-7.5	0.92	-2.1	-258	0.42	3.6	
B	3	-214	-1.5	0.78	-276	-73.9	-7.0	-237	-10	0.86	-1.9	-268	0.56	5.3	15M, Straight rebar
	4	-212	-1.5	0.64	-334	-87.1	-8.0	-292	-9.5	0.87	-2.1	-300	0.53	4.5	
C	5	-173	-1.5	0.80	-216	-52.6	-4.0	-181	-8.0	0.84	-1.8	-202	0.45	4.8	10M, 180° hook
	6	-215	-2.0	1.00	-215	-52.6	-2.0	-144	-8.0	0.67	-1.6	-170	0.45	4.4	
D	7	-175	-1.5	0.84	-208	-50.8	-4.5	-182	-8.5	0.87	-1.7	-200	0.47	5.0	10M, 90° hook
	8	-166	-1.5	0.77	-217	-52.9	-3.5	-162	-9.0	0.75	-1.7	-193	0.50	4.9	
E	9	-158	-1.5	0.74	-214	-52.1	-5.0	-158	-9.0	0.74	-1.9	-201	0.50	4.6	10M, Shear studs
	10	-150	-1.5	0.81	-184	-45.0	-4.5	-156	-7.5	0.85	-1.7	-173	0.42	4.3	
F	11	-144	-1.5	0.80	-180	-30.0	-3.0	-139	-8.0	0.77	-1.7	-167	0.45	4.6	10M, 90° hook, BB: 3 rd and 6 th
	12	-147	-1.5	0.71	-206	-40.0	-5.0	-134	-8.5	0.65	-1.9	-185	0.47	4.5	
G	13	-143	-1.5	0.66	-215	-43.0	-6.0	-170	-7.5	0.79	-2.9	-213	0.42	2.6	10M, 90° hook, 5 th course reinforced
	14	-147	-1.5	0.71	-206	-40.0	-3.5	-153	-8.0	0.74	-1.9	-187	0.45	4.2	

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Table 5: Displacements at each Strain Gauge when Yielding Occurred

Group	Wall	SG1 _{dy} (mm)	SG2 _{dy} (mm)	SG3 _{dy} (mm)	SG4 _{dy} (mm)	SG5 _{dy} (mm)	SG6 _{dy} (mm)	SG7 _{dy} (mm)	SG8 _{dy} (mm)	SG9 _{dy} (mm)	SG10 _{dy} (mm)	SG11 _{dy} (mm)	SG12 _{dy} (mm)	d _{@H_{max}} (mm)*	Yielding Remark @ Exterior SGs
A	1	-	-5	-5.5	-5	-5.5	-5	-5.5	-5	-				3	Yielding @ H _{max}
														-5.5	
	2	-	5.5	-	4	4.5	6	5.5	-	-				3	Yielding after H _{max}
														-7	
B	3	9	-	-	3.5	6	5	-	-	-				4	Yielding after H _{max} †
														-7	
	4	9.5	-	5.5	4.5	7	-	-	-	-				9	Yielding after H _{max}
														-8	
C	5	7.5	-				-8.5	-7.5	-	-				4	Yielding after H _{max}
														-4.0	
	6	5.5	4.5				4	-	-	-				4.5	Yielding after H _{max}
														-2	
D	7	6	6				-	-	-	-				5	Yielding after H _{max}
														-4.5	
	8	7	-				-	-	-	-				4.5	Yielding after H _{max}
														-3.5	
E	9	-	9				-	-	-	-				6.5	Yielding after H _{max}
														-5	
	10	-	-				-	6.5	-	-				5	Yielding after H _{max}
														-4.5	
F	11	-	-	-	-	-	-	-	-	-	-	-	-	5	No yielding
														-3	
	12	-	-	-	-	-	-	-	-	-	-3	-	-	4.5	Yielding before H _{max} only at one location
														-5.0	
G	13	-	7.5	-		-	-	5.5						6	Yielding after/before (one location) H _{max}
														-6	
	14	-	-	6.5		-5.5	-7	-						5.5	Yielding @ and after H _{max}
														-3.5	

* The displacement at which maximum load has occurred is bolded.

†Yielding at the location of more critical strain gauges, where development length can be of concern.

Table 6: Proportion of bars yielding at the Strain Gauge Locations by Bar Size

Set	SG1	SG2	SG6	SG7	AVG
15M	0.50	0.50	0.75	0.50	0.56
10M	0.67	0.50	0.33	0.50	0.50

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- Figure 4: Vertical Load Adjustment (a) Wall and Applied Loads (b) Zero Moment at Mid-height (Groups A and B) (c) Zero Moment 415 mm Below the Top of the Wall (The rest of the walls)
- Figure 5: Bilinear Idealized Curve vs. Experimental Envelope
- Figure 6: Wall Behaviour in Group A (15M -180° Hooks) and B (15M-Straight Rebar) (a) W1 (b) W2 (c) W3 (e) W4 (Lines in red and green are the hysteresis envelope and bilinear idealized curves respectively)
- Figure 7: Wall Behaviour in Group C (10M-180° Hooks), D (10M-90° Hooks) and E (10M-Shear Studs) (a) W5 (b) W6 (c) W7 (d) W8 (e) W9 (f) W10
- Figure 8: Wall Behaviour in Group F (10M-90° Hooks, Bond Beam in 3rd & 9th Courses) and G (10M-90° Hooks in 5th Course, Grouted 9th Course) (a) W11 (b) W12 (c) W13 (d) W14
- Figure 9: Normalized Load-Displacement Envelopes
- Figure 10: Strains Recorded in Rebars in Wall 4 (a) Strain Recorded by SGs Located at the Middle of the Rebar in Course 5 (b) Strain Recorded by the SGs Located at the Ends of the Rebars
- Figure 11: Average Displacement Ductility by Group (a) Push (b) Pull

Figure 12: Average Effective Stiffness in Push and Pull Directions (a) Experimental (b)

Normalized

Figure 13: Energy in One Loading Cycle (a) Input Energy (b) Dissipated Energy

Figure 14: Dissipated Energy at Different Stages

Fig. A1: Crack Pattern in Walls in Group A (15M -180° Hooks) and B (15M-Straight Rebar) (a)

W1 (b) W2 (c) W3 (e) W4

Fig. A2: Crack Pattern in Walls in Group C (10M-180° Hooks), D (10M-90° Hooks) and E

(10M-Shear Studs) (a) W5 (b) W6 (c) W7 (d) W8 (e) W9 (f) W10

Fig. A3: Crack Pattern in Walls in Group F (10M-90° Hooks, Bond Beam in 3rd & 9th Courses)

and G (10M-90° Hooks in 5th Course, Grouted 9th Course) (a) W11 (b) W12 (c) W13 (d) W14

Appendix A

Crack pattern for all the walls are presented in this section. The crack pattern of wall one was significantly different from the other walls as it failed in a brittle manner like an unreinforced wall, probably because of the problems with the very first test. Other than that, the cracking patterns of all the walls were similar. Walls with different horizontal rebar distribution and end anchorage conditions all experienced distributed cracks across the wall mostly in the lower courses with fewer cracks in the top courses and only a few cracks in the topmost course.

Table 1: Wall Specimens and the Variable Parameters

Group	Wall Number	Bond Beam Course #	Horizontal Rebar Spacing (mm)	Horizontal Reinforcement Size (Ratio)	Anchorage Type
A	1,2	5,9	800	15M* (0.12%)	180°
B	3,4				Straight
C	5,6			10M† (0.06%)	180°
D	7,8				90°
E	9,10				Shear Stud
F	11,12	3,6	600		90°
G	13,14	5,9 (unreinforced)	NA	10M (0.03%)	90°

* 15M = 16 mm diameter

† 10M = 11.3 mm diameter

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Table 3: Test Results/Calculated Values in Push Direction

Group	Wall	Crack			Maximum			Failure			Bilinear Idealized Values					
		H _{cr} (kN)	d _{cr} (mm)	H _{cr} /H _{max}	H _{max} (kN)	Normalized H _{max}	d _{@Hmax} (mm)	H _{@Dmax} (kN)	d _{max} (mm)	H _{@Dmax} /H _{max}	d _e (mm)	H _u (kN)	drift (%)	μ _d (mm/mm)	P _{@Hmax} (kN)	Wall Remark
A	1	194	1.0	0.84	232	62.1	3.0	170	5.5	0.73	1.1	218	0.31	4.9	449	15M, 180° hook
	2	227	1.0	0.70	327	87.3	3.0	236	7.5	0.72	1.2	274	0.42	6.2	499	
B	3	264	1.5	0.76	346	92.5	4.0	278	10.0	0.80	1.8	321	0.56	5.5	517	15M, Straight rebar
	4	238	1.5	0.84	282	76.8	9.0	251	9.5	0.89	1.6	259	0.53	5.8	505	
C	5	185	1.5	0.80	232	56.5	4.0	182	8.0	0.79	1.8	218	0.45	4.5	409	10M, 180° hook
	6	169	1.5	0.89	191	46.6	4.5	177	8.0	0.93	1.6	186	0.45	4.9	409	
D	7	194	1.5	0.79	244	59.5	5.0	190	8.5	0.78	1.8	228	0.47	4.8	409	10M, 90° hook
	8	184	1.5	0.83	223	54.5	4.5	200	9.0	0.90	1.8	220	0.50	5.0	409	
E	9	147	1.5	0.65	226	55.1	6.5	157	9.0	0.70	2.1	209	0.50	4.2	409	10M, Shear studs
	10	177	1.5	0.83	213	51.9	5.0	144	7.5	0.68	1.6	193	0.42	4.6	409	
F	11	172	1.5	0.81	211	42.0	5.0	167	8.0	0.79	1.7	198	0.45	4.6	409	10M, 90° hook, BB: 3 rd and 6 th
	12	152	1.5	0.73	209	41.0	4.5	152	8.5	0.73	2.0	197	0.47	4.4	409	

Table 2: Materials Compressive Strengths

Group	Masonry Block (MPa) Mean±SD	Mortar (MPa) Mean±SD	Grout (MPa) Mean±SD	Fully Grouted Prism (f _{gm}) (MPa) Mean±SD	Hollow Prism (f _{hm}) (MPa) Mean±SD	Weighted Prism Strength (f _{mw}) (MPa)*	Characteristic Compressive Strength (f _m) † (MPa)	Age (Days)
A	35.7 ±3.5	22.1 ±3.3	25.7 ±2.4	10.5 ±2.3	18.6 ±2.8	13.3	10.0	30
B			W4: 21.7 ±2.3	W4: 9.5 ±2		W4: 12.7	W4: 9.8	
C	29.3 ±2.1	9.5 ±1.1	25.2 ±6.3	12.7 ±2.4	22.3 ±2.2	16.0	13.0	28
D		11.5 ±0.6	31.7 ±2.1					
E		11 ±2.4	31.8 ±3.6					
F	20.1 ±2.2	9.3 ±1	24.7 ±1.7	8.8 ±1.2	14.7 ±0.6	10.8	9.9	
G		11.6 ±2.4	29.3 ±1.8					

* $f'_{mw} = \frac{\frac{1}{3} \cdot f'_{gm} \cdot 0.9 \cdot 190 \cdot 1000 + \frac{2}{3} \cdot f'_{hm} \cdot 0.9 \cdot 75500}{\frac{1}{3} \cdot 190 \cdot 1000 + \frac{2}{3} \cdot 75500}$ in which 75500 mm² is the effective mortared area for one meter of 190 mm concrete block wall and 0.9 height to thickness correction factor for prism strength is based on clause D.6.5, table D.1 in annex D of CSA-S304-14 (2014).

† $f'_m = f'_{mw} - 1.64 \text{ SD}$, so the actual compressive strength exceeds the specified compressive strength 95% of the time according to Annex C of CSA S304-14 (2014)

G	13	151	1.5	0.86	175	28.0	6.0	162	7.5	0.93	1.7	172	0.42	4.4	409	10M, 90° hook, 5 th course reinforced
	14	156	1.5	0.68	230	49.0	5.5	174	8.0	0.76	2.0	212	0.45	5.3	409	

H_{cr} (kN): Horizontal load when the first noticeable crack occurs

d_{cr} (mm): first noticeable crack

H_{cr} / H_{max}

H_{max} (kN): Maximum lateral load tolerated by the wall

Normalized H_{max} : Normalized maximum lateral load

$d_{@Hmax}$ (mm): Displacement at the maximum lateral load

$H_{@Dmax}$ (kN): Horizontal load at the maximum displacement

d_{max} (mm): Maximum displacement

$H_{@Dmax} / H_{max}$: Ratio of horizontal load at the maximum displacement to the maximum horizontal load

d_e (mm): Elastic displacement

H_u (kN): Ultimate horizontal load

drift (%): Ratio of displacement

μ_d (mm/mm): ductility ratio which is d_u/d_e

$P_{@Hmax}$ (kN): Axial load at maximum shear load

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Table 4: Test Results/Calculated Values in Pull Direction

Group	Wall	Crack			Maximum			Failure			Bilinear Idealized Values				
		H_{cr} (kN)	d_{cr} (mm)	H_{cr}/H_{max}	H_{max} (kN)	Normalized H_{max}	$d_{@H_{max}}$ (mm)	$H_{@D_{max}}$ (kN)	d_{max} (mm)	$H_{@D_{max}}/H_{max}$	d_e (mm)	H_u (kN)	drift (%)	μ_d (mm/mm)	Wall Remark
A	1	-182	-1.0	0.67	-273	-73.0	-5.5	-273	-5.5	1.00	-1.2	-226	0.31	4.4	15M, 180° hook
	2	-184	-1.5	0.67	-276	-73.8	-7.0	-255	-7.5	0.92	-2.1	-258	0.42	3.6	
B	3	-214	-1.5	0.78	-276	-73.9	-7.0	-237	-10	0.86	-1.9	-268	0.56	5.3	15M, Straight rebar
	4	-212	-1.5	0.64	-334	-87.1	-8.0	-292	-9.5	0.87	-2.1	-300	0.53	4.5	
C	5	-173	-1.5	0.80	-216	-52.6	-4.0	-181	-8.0	0.84	-1.8	-202	0.45	4.8	10M, 180° hook
	6	-215	-2.0	1.00	-215	-52.6	-2.0	-144	-8.0	0.67	-1.6	-170	0.45	4.4	
D	7	-175	-1.5	0.84	-208	-50.8	-4.5	-182	-8.5	0.87	-1.7	-200	0.47	5.0	10M, 90° hook
	8	-166	-1.5	0.77	-217	-52.9	-3.5	-162	-9.0	0.75	-1.7	-193	0.50	4.9	
E	9	-158	-1.5	0.74	-214	-52.1	-5.0	-158	-9.0	0.74	-1.9	-201	0.50	4.6	10M, Shear studs
	10	-150	-1.5	0.81	-184	-45.0	-4.5	-156	-7.5	0.85	-1.7	-173	0.42	4.3	
F	11	-144	-1.5	0.80	-180	-30.0	-3.0	-139	-8.0	0.77	-1.7	-167	0.45	4.6	10M, 90° hook, BB: 3 rd and 6 th
	12	-147	-1.5	0.71	-206	-40.0	-5.0	-134	-8.5	0.65	-1.9	-185	0.47	4.5	
G	13	-143	-1.5	0.66	-215	-43.0	-6.0	-170	-7.5	0.79	-2.9	-213	0.42	2.6	10M, 90° hook, 5 th course reinforced
	14	-147	-1.5	0.71	-206	-40.0	-3.5	-153	-8.0	0.74	-1.9	-187	0.45	4.2	

Table 5: Displacements at each Strain Gauge when Yielding Occurred

Group	Wall	SG1 _{dy} (mm)	SG2 _{dy} (mm)	SG3 _{dy} (mm)	SG4 _{dy} (mm)	SG5 _{dy} (mm)	SG6 _{dy} (mm)	SG7 _{dy} (mm)	SG8 _{dy} (mm)	SG9 _{dy} (mm)	SG10 _{dy} (mm)	SG11 _{dy} (mm)	SG12 _{dy} (mm)	d _{@H_{max}} (mm)*	Yielding Remark @ Exterior SGs
A	1	-	-5	-5.5	-5	-5.5	-5	-5.5	-5	-				3	Yielding @ H _{max}
														-5.5	
	2	-	5.5	-	4	4.5	6	5.5	-	-				3	Yielding after H _{max}
														-7	
B	3	9	-	-	3.5	6	5	-	-	-				4	Yielding after H _{max} †
														-7	
	4	9.5	-	5.5	4.5	7	-	-	-	-				9	Yielding after H _{max}
														-8	
C	5	7.5	-				-8.5	-7.5	-	-				4	Yielding after H _{max}
														-4.0	
	6	5.5	4.5				4	-	-	-				4.5	Yielding after H _{max}
														-2	
D	7	6	6				-	-	-	-				5	Yielding after H _{max}
														-4.5	
	8	7	-				-	-	-	-				4.5	Yielding after H _{max}
														-3.5	
E	9	-	9				-	-	-	-				6.5	Yielding after H _{max}
														-5	
	10	-	-				-	6.5	-	-				5	Yielding after H _{max}
														-4.5	
F	11	-	-	-	-	-	-	-	-	-	-	-	-	5	No yielding
														-3	
	12	-	-	-	-	-	-	-	-	-	-3	-	-	4.5	Yielding before H _{max} only at one location
														-5.0	
G	13	-	7.5	-		-	-	5.5						6	Yielding after/before (one location) H _{max}
														-6	
	14	-	-	6.5		-5.5	-7	-						5.5	Yielding @ and after H _{max}
														-3.5	

* The displacement at which maximum load has occurred is bolded.

†Yielding at the location of more critical strain gauges, where development length can be of concern.

Table 6: Proportion of Bars Yielding at the Strain Gauge Locations by Bar Size

Set	SG1	SG2	SG6	SG7	AVG
15M	0.50	0.50	0.75	0.50	0.56
10M	0.67	0.50	0.33	0.50	0.50

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Figure 1: Wall Construction (a) Wall Specimen (b) 180° Hook Horizontal Rebar Placement (c) Studded Rebar (d) Studded Rebar Placement

(a)

(b)

(c)

(d)

Figure 2: Reinforcement and Strain Gauge Placement in the Different Groups of Walls (a) Groups A and B (b) Groups C, D and E (c) Group F (d) Group G

Figure 3: In-Plane Shear Test Arrangement

Figure 4: Vertical Load Adjustment (a) Wall and Applied Loads (b) Zero Moment at Mid-height (Groups A and B) (c) Zero Moment 415 mm Below the Top of the Wall (The rest of the walls)

Figure 5: Bilinear Idealized Curve vs. Experimental Envelope

Figure 6: Wall Behaviour in Group F (10M-90° Hooks, Bond Beam in 3rd & 9th Courses) and G (10M-90° Hooks in 5th Course, Grouted 9th Course) (a) W11 (b) W12 (c) W13 (d) W14

Figure 7: Normalized Load-Displacement Envelopes

(a)

(b)

Figure 8: Strains Recorded in Rebars in Wall 4 (a) Strain Recorded by SGs Located at the Middle of the Rebar in Course 5 (b) Strain Recorded by the SGs Located at the Ends of the Rebars

Figure 9: Normalized Load-Displacement Envelopes

(a)

(b)

Figure 10: Strains Recorded in Rebars in Wall 4 (a) Strain Recorded by SGs Located at the Middle of the Rebar in Course 5 (b) Strain Recorded by the SGs Located at the Ends of the Rebars

Figure 11: Average Displacement Ductility by Group (a) Push (b) Pull

Figure 12: Average Effective Stiffness in Push and Pull Directions (a) Experimental (b) Normalized

Figure 13: Energy in One Loading Cycle (a) Input Energy (b) Dissipated Energy

Figure 14: Dissipated Energy at Different Stages

Fig. A1: Crack Pattern in Walls in Group A (15M -180° Hooks) and B (15M-Straight Rebar) (a) W1 (b) W2 (c) W3 (e) W4

(a)

(b)

(c)

(d)

Fig. A2: Crack Pattern in Walls in Group C (10M-180° Hooks), D (10M-90° Hooks) and E (10M-Shear Studs) (a) W5 (b) W6 (c) W7 (d) W8 (e) W9 (f) W10

Fig. A3: Crack Pattern in Walls in Group F (10M-90° Hooks, Bond Beam in 3rd & 9th Courses) and G (10M-90° Hooks in 5th Course, Grouted 9th Course) (a) W11 (b) W12 (c) W13 (d) W14