EXPERIMENTAL EVALUATION OF THE SHEAR CAPACITY OF REINFORCED MASONRY SHEAR WALLS

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ABSTRACT

International masonry design standards differ significantly in treatment of the parameters affecting shear capacity of reinforced masonry shear walls. As a result, the predicted capacities exhibit various degrees of conservatism relative to experimental values. Since the masonry material and construction practices are similar in these compared countries, the discrepancies point to different interpretations of the importance of each parameter and, perhaps, different philosophies related to conservatism and accuracy requirement for shear design. The provisions in CSA S304.1-04, Masonry Design for Buildings [1] are among the most conservative.

This paper contains the results of cyclic tests of 4 reinforced masonry shear walls designed to fail in shear. The parameters studied include amount of shear reinforcement, presence of axial load, masonry compressive strength, and wall size. The results indicate that the Canadian formulations are very conservative and that shear reinforcement is much more effective than predicted. Despite failing in a shear mode, the test walls exhibited significant post-peak capacity and ability to dissipate energy under reversed cyclic loading.

KEYWORDS: Shear walls, Reinforced masonry, Concrete blocks, Design equations, Cyclic test

INTRODUCTION

The shear resistance of masonry, or even of concrete for that matter, is not fully understood and is difficult to realistically model. Large differences exist in international design standards [1,2,3,4,5] regarding appropriate design parameters and the relative conservatism of the design. Because flexural resistance is well understood, such elements are efficiently designed and the materials are used to their fullest potential. The opposite seems to be the case for shear resistance, where over-design and inefficient use of materials result.

Some of the observed differences can be attributed to different research results and to different interpretations. In addition, the assessment of the importance of having accurate representations of shear capacity versus ensuring adequate shear capacity depends on the design culture and, sometimes, special conditions such as earthquake controlled design. Nonetheless, since a particular shear wall's resistance is not affected by national borders, the great variations point to some lack of consensus on shear capacity.

Review of the different code approaches suggests that the horizontal shear reinforcement is underutilized in Canadian design [1]. The equation in CSA S304.1-04, (Section 10.11.1) for 'Factored in-plane shear resistance' of walls is as follows;

$V_r = \varphi_m(v_m\, b_w\, d_v + 0.25\, P)\gamma_g + \varphi_s\,(0.60\, A_v\, f_y\, d_v/s)$, Equation 1

but not greater than,

$V_r = (0.4)\varphi_m \sqrt{f'_m}\; b_w\, d_v\, \gamma_g$, Equation 2

where,

$v_m = 0.16(2 - M_f/V_f d_v)\sqrt{f'_m}$ Equation 3

φ_m = material resistance factor for masonry, = 0.60
φ_s = material resistance factor for steel, = 0.85
b_w = width of the web of the shear wall
d_v = effective depth which need not be taken less than $0.8 l_w$ for walls with flexure reinforcement distributed along the length
l_w = length of wall
P = axial compressive load on the section under consideration
γ_g = factor to account for partially grouted walls, = 1.0 for fully grouted walls
A_v = area of individual shear reinforcement
M_f = factored moment at the section under consideration
V_f = factored shear at the section under consideration
f_y = yield strength of shear reinforcement
s = spacing of shear reinforcement
f'_m = compressive strength of masonry

It can be seen that the traditional effectiveness of the horizontal shear reinforcement has been reduced to sixty percent, which is further reduced to forty-eight percent due to the effective depth being taken as 0.8 of the wall length. Although not proven, it has been speculated [7,8] that a reason for discounting the effectiveness of the shear reinforcement is that research data supports such a reduction where inadequate anchorage of the reinforcement caused premature failure.

EXPERIMENTAL PROGRAM

A preliminary series of four full scale shear wall tests was arranged to provide some initial insight into the factors identified in Equation 1 as affecting shear strength. Therefore, as indicated in Table 1, masonry strength, shear to moment capacity relationship (related to wall size), percentages of reinforcement in both the vertical and horizontal direction, and presence of axial compressive load were included.

Since the purpose of this study was to evaluate shear resistance, the four test walls were designed to fail in shear. This was accomplished by calibrating the Canadian design equation to shear wall data by Shing et al. [9]. As a result, the four walls required relatively high amounts of vertical reinforcement to avoid having flexural failure occur prior to shear failure. (Of course this is opposite to what is desired in design practice.)

Table 1 – Wall Specifications

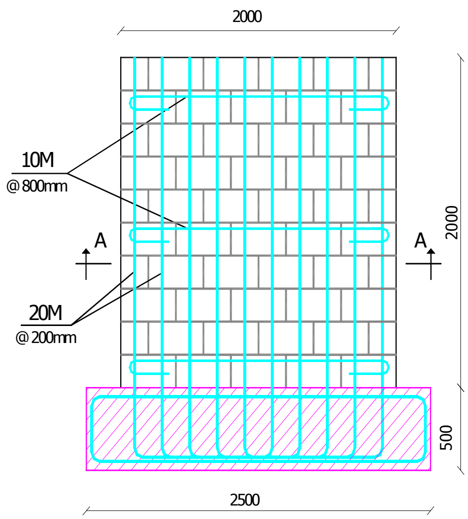
Wall Number	Masonry Strength f'_m (MPa)	Wall Dimensions Length (m)	Height (m)	Vertical Reinforcement A_v (mm^2)	ρ_v (%)	Horizontal Reinforcement Spacing (mm)	ρ_h (%)	Axial Load P (MPa)
1	15.4	2	2	300	0.79	800	0.07	1
2	12.7	2	2	300	0.79	400	0.13	0
3	15.4	3	3	300	0.79	400	0.13	1
4	12.7	3	3	300	0.79	800	0.07	0

WALL CONSTRUCTION

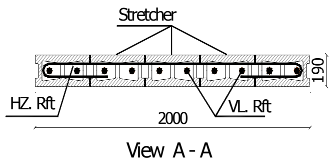
A qualified mason constructed each wall on its own disposable reinforced concrete base. As shown in Figure 1(a), the 500mm high base had a width of 520mm and extended beyond the ends of the test walls. The vertical reinforcement was carefully positioned in the concrete forms to ensure adequate anchorage and correct positioning within the subsequently built concrete block wall. In order to avoid complications associated with using lap splices, the vertical reinforcement was placed in single lengths over the full wall height. 20M bars were placed in each cell, as shown in Figure 1. The use of pre-positioned vertical bars without splices required that the standard stretcher units be threaded down over the top of the bars. While this was an inconvenient method of construction, it was considered necessary to avoid introducing additional factors that might obscure the fundamental behaviour.

10M bars were used as horizontal reinforcement and were placed in either every second or every fourth course. As shown in Figure 1(c), the webs of the concrete block stretcher units were saw cut to half the block height to create knock-out web blocks. These blocks were used for the masonry courses containing horizontal reinforcement and had the advantages of having the same properties as the block in other courses. As can be seen in Figure 1(d), to provide the adequate anchorage, the horizontal reinforcement was hooked 180° around the outside vertical bar and extended back 400mm. As mentioned by others [7], lack of adequate anchorage of shear reinforcement in practice and in wall tests may be one of the reasons that effectiveness of shear reinforcement has been discounted. Also, through the use of Equation 2, limits have been imposed on maximum amounts of shear steel that can be used effectively in shear capacity calculations.

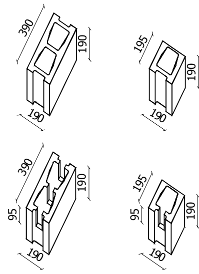
After the mortar had set, coarse grout from a ready mix plant was delivered to the laboratory and pumped into place. The walls were filled in one nearly continuous lift with only minor pauses required to consolidate the grout. Despite having an initial 235 mm grout slump, concerns about congestion of reinforcement, particularly in the vicinity of the hooked horizontal reinforcement, led to taking special care to vibrate the grout into place. A pencil vibrator with a 25 mm diameter head was used except, for the lower regions of the walls that could not be reached by the vibrator, consolidation was actually achieved by vibrating the reinforcement which, in turn, vibrated the grout.



a) Elevation of Wall 1 showing reinforcing details



b) Cross-section of Wall #1



c) 20 cm stretcher units

d) Knock-out web blocks with vertical and horizontal reinforcement

Figure 1 – Construction Details for Test Walls

After completing the tests, dismantling of the test walls showed that all of the block cells were completely filled with grout to form continuous well compacted grout columns. However, for the discontinuous cells in the head joint at the frogged ends of the blocks, about ⅓ were incompletely filled with grout (Figure 2(a)), which was sometimes also poorly compacted (Figure 2(b)). This experience causes some concern for use of coarse grout to achieve solid masonry where standard units create isolated cells between the ends of blocks. It should be noted that delays and warm weather caused significant loss of fluidity during construction.

a) Partial grouting of head joint cell

b) Poorly compacted grout

Figure 2 – Incomplete grouting of cells at frogged ends of blocks

TEST SETUP

Figure 3 contains a drawing of the test setup. As can be seen, the reinforced concrete base for the wall was prestressed down onto a 600 mm thick floor beam which had previously been poured in place and prestressed down to the 600 mm thick strong floor in the Applied Dynamics Laboratory. Care was taken to ensure that the wall was plumb and aligned with the hydraulic actuator.

To ensure that the lateral in-plane load (shear force) was applied uniformly along the length of the wall, a reusable steel beam was attached to the top of the wall. Holes drilled through the base of this built up U-shaped beam coincided with locations of the vertical reinforcement so that extensions of these bars above the top of the wall could be welded to this loading beam.

The attachment of the hydraulic actuator to the beam was offset so that the line load coincided with the top of the wall. As shown in Figure 3, tension rods were used to similarly apply the load to the far end of the loading beam for the pull cycle of loading. This allows for the single actuator to literally push the test wall from either side. The loading beam also provided a convenient way to stabilize the wall against out-of-plane instability as the loading exceeded the in-plane capacity. Braces consisted of rigid arms extending from columns in the laboratory. These were equipped with roller bearings that fit onto pintels extending down between the legs of the U in the loading beam. The loading beam also helped distribute axial compressive loading for the walls chosen to investigate this effect.

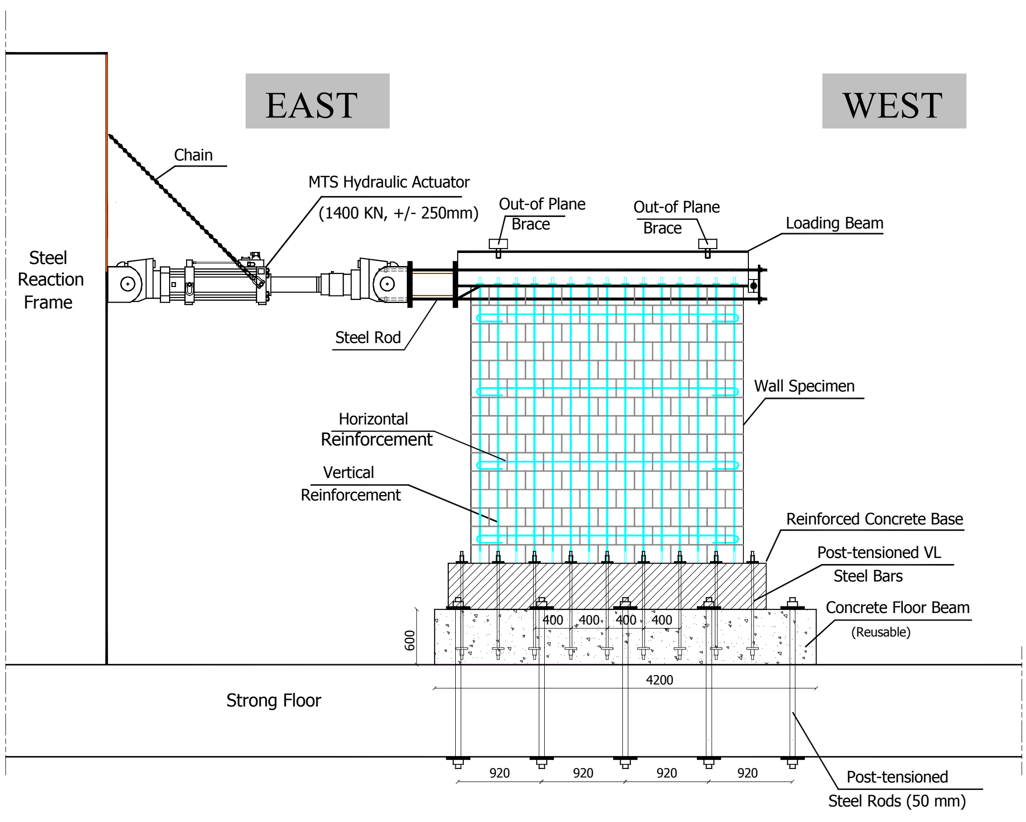
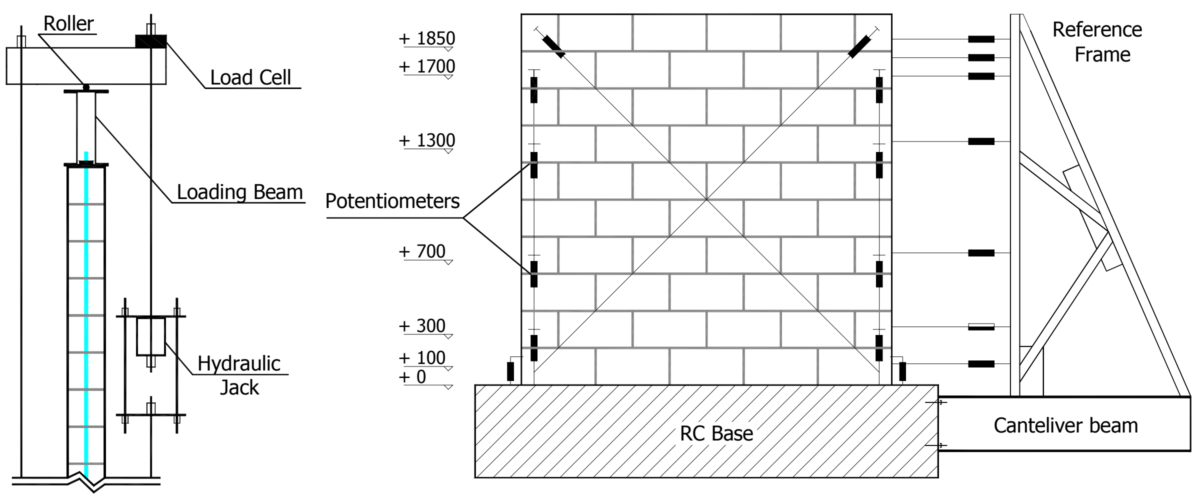


Figure 3 – Test Setup

Of particular note in the above test setup was the use of a cast in-place floor beam to serve as the attachment point for the base of the test wall. This element was cast in place so that it fit the contours of the laboratories floor. Prestressing it to the laboratory floor on 0.91 m centres, using 63.5 mm diameter threaded bolts equipped with doughnut shaped spacers, avoided any slip displacement or rotation of the test apparatus.

High strength threaded prestressing bars were used to prestress the wall bases onto the floor beam to again avoid slip or rotation at the base of the test wall. Appropriate sleeves were cast into both elements to accommodate this prestressed anchorage.

An axial load that produces a compressive stress of 1 MPa was applied through the use of two hollow cylinder hydraulic jacks as shown in Figure 4(a). The roller at the top of the loading beam ensures an equal force in each of the rods. The two point loads were monitored with load cells and independently adjusted throughout the tests as bending of the wall caused changes in their forces. As can be seen in Figure 4(a), inverting the hydraulic jack allowed compressive force on the jack to apply tension to the rods that transfer compression onto the top of the loading beam. These rods were high strength prestressing steel, anchored into the floor beam. A small adjustment to the magnitude of the applied horizontal in-plane shear force was required to account for a horizontal component of the axial load as the wall deflected during lateral loading.



a) Application of Axial Load b) Instrumentation (for 2m high walls)

Figure 4 – Axial Loading Setup and Instrumentation Arrangement

Another element used in the test setup was a reference frame, rigidly connected to the wall base as shown in Figure 4(b). Having such a frame attached to the base of the wall, eliminated potential problems associated with sliding or rotation of the base, with respect to the laboratory floor. Horizontal linear potentiometric displacement transducers were attached to the reference frame and to the wall at various heights. In addition, to be able to calculate flexural and shear displacements, vertical and diagonal potentiometers were placed directly on the wall between two common anchors. These strain rosettes were necessary to distinguish between shear and flexural deformations.

MATERIALS

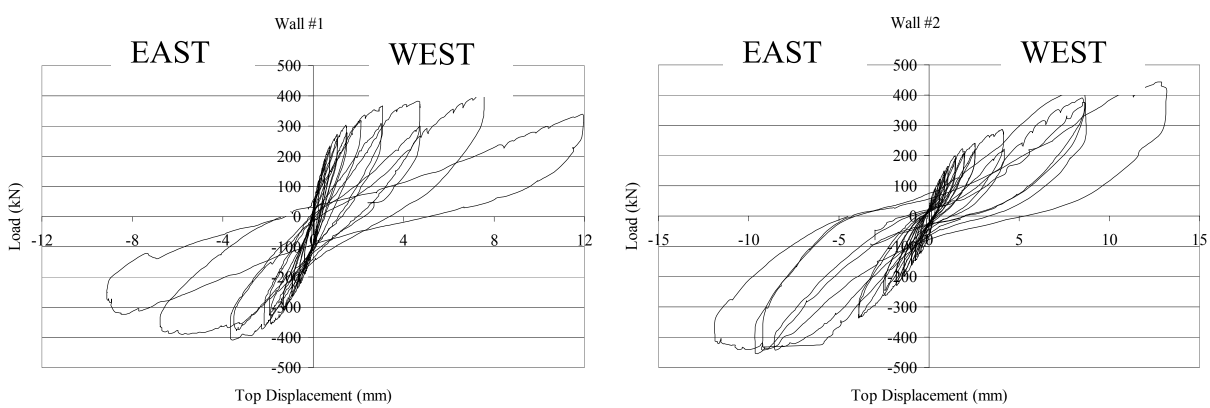
The intent was to construct walls using two different strengths of 20 cm hollow concrete block in the hope of producing f'm values of about 12 and 20 MPa. However, the two runs of blocks used for construction were closer in strength than anticipated. The lower strength block to be used within the walls calling for 12 MPa masonry strength ended up having a compressive strength of 21.8 MPa, while the higher strength block for use in 20 MPa walls were 25.7 MPa. A Type S mortar, with an average compressive strength of 22.7 MPa, from cube tests, was used to construct the walls, as well as eighteen four-course high prisms. The mortar had proportions by weight of Portland cement : lime : dry masonry sand of 1 : 0.21 : 3.53.

All walls and prisms were grouted at the same time. The coarse grout had an average 28 day cylinder compressive strength of 26.8 MPa. From tests of nine, four-course high grouted prisms for each of the 2 block strengths, the average masonry compressive strengths were 12.7 and 15.4 MPa. The closeness of these values due to similar block strengths reduced the ability to study effects of compressive strength of masonry.

The 10M horizontal reinforcement had a yield stress of 425 MPa.

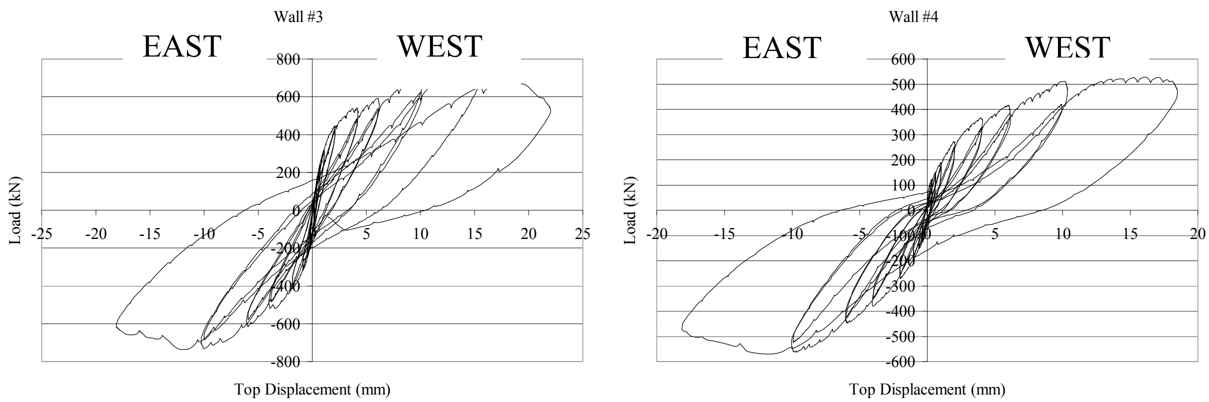
SHEAR WALL TESTS

The walls were initially loaded to 20 % of the expected capacity based on Shing et al test results [9]. Then cyclic testing was continued using displacement control with two cycles at each displacement. Displacement increments were gradually increased as illustrated in Figures 5 and 6. As can be seen in these figures, the hysteresis loops are quite symmetric up to failure. Failure was deemed to have occurred when the load decreased for an increase in displacement. At failure, the response in the first loaded direction (West) appears to be slightly more ductile than when loaded back in the other direction. Possibly this is an indication of the effect of damage during the failure load cycle. As can be seen, despite extensive visible damage, significant post-peak deformability remained with relatively little degradation of capacity for large increments of deflection. The shear failures were neither sudden nor brittle.



a) Hysteresis Response for Wall 1　　b) Hysteresis Response for Wall 2

Figure 5 - Load vs. Top Displacements for 2m x 2m Walls



a) Hysteresis Response for Wall 3 b) Hysteresis Response for Wall 4

Figure 6 - Load vs. Top Displacements for 3m x 3m Walls

Brief comments are provided below for each wall.

Wall 1. In common with Wall 3, maintaining the axial load using two independent jacks required continuous adjustment as tensile and compressive deformations occurred due to bending of the wall. For this wall, substantial diagonal “X” type cracking was visible at 300 kN lateral load. This followed initial flexural cracking along bed joints and smaller diagonal cracks following a path through a combination of head and bed joints. As can be seen in Figure 5(a), stiffness decreased significantly at lateral loads above 300 kN and larger cracks, passing through blocks were observed as shown in Figure 7(a). As the load approached its maximum value for loading in the East direction, the “toe” on the East side of the wall began to crush as shown in Figure 7(a). From this point on, not many new cracks formed but the main diagonal “X” cracks became much wider. With displacement controlled loading, there was no sudden change in behaviour.

Wall 2. The behaviour of Wall 2 was very similar to Wall 1 with the net effect of no superimposed axial compression and doubled shear reinforcement producing about 10 % increase in shear capacity. Unfortunately, after the wall had reached its maximum load for loading in the West direction, no further loading was carried out in that direction to document the degradation of capacity with damage due to cyclic repetitions of deflection. The reason was that the wall was beginning to be pushed out-of-plane causing large forces on the out-of-plane bracing. Reloading in the more stable East direction resulted in the load cycle beyond maximum load.

Wall 3. This wall displayed very small mortar joint cracks for displacements up to 2.5 mm corresponding to lateral load of nearly 500 kN. At approximately 550 kN, cracks began to cross through blocks rather than follow mortar joints. Near maximum load, vertical end cracks began to propagate through the webs on both sides of the block cell from the base up seven courses. Increased displacement loading produced delamination of the face shells from the grouted cores of the wall. As shown in Figure 6(a), the hysteresis loops for the final loading cycles were not symmetric. Loss of a displacement potentiometer attachment point caused a 2.5 mm additional

East direction displacement. Again, relatively ductile behaviour with a relatively small amount of capacity degradation was observed.

Wall 4. The behaviour of Wall 4 was very similar to Wall 3. As shown in Figure 6(b), symmetry was observed over the length of the test. Face shell delamination over most of the lower corners of the wall was observed near maximum/post-maximum loading. Nonetheless, large post-peak deformations were observed corresponding to relatively small post-peak degradation of capacity. Figure 7(b) shows well developed diagonal cracking at maximum capacity for loading in the West direction.

a) Wall 1, East Toe

b) Wall 8, West Toe

Figure 7 - Diagonal Crack Photographs

The ultimate loads reached in each direction for cyclic loading in the West and East direction are listed in Table 2 for all walls. Also, two capacities predicted using modified versions of Equation 1 are listed for each wall. The first capacity was calculated setting material resistance factors $\varphi_m = \varphi_s = 1.0$ and using actual material strengths. The second capacity used an additional modification by using a 1.0 factor instead of the 0.6 factor for the shear reinforcement (A_v) part of the equation. The ratios of these predicted capacities to the least East/West test value are also shown. As can be seen, with the material resistance factors removed and actual strengths of materials, the code (1) varies from being 17 – 38 % conservative. If the shear reinforcement is considered to be 100 % effective, this conservatism changes to 7 – 23 %. It is noteworthy that predictions were much closer to test capacities for walls with axial load applied.

Table 2 – Ultimate Shear Resistance of Walls

Wall Number	Ultimate Shear Force (kN)			CSA S304.1 Predicted V_r (kN)			
	Loading West	Loading East	Least of East/West	Eq. 1*	Eq. 1* / Least Ultimate	Eq. 1 but with Av 100% Effective**	Eq. 1** / Least Ultimate
1	408	409	408	337	0.83	371	0.91
2	443	454	443	275	0.62	343	0.77
3	774	735	735	582	0.79	684	0.93
4	528	570	528	337	0.64	388	0.73

* Calculated with material resistance factors equal to 1.0 and actual material strengths.

** Same as *, however, the 0.6 coefficient before the steel component was set equal to 1.0.

CONCLUSIONS

These initial shear wall test results indicate that the current Canadian approach (1) is conservative. However, the basic formulation provides a reasonable and less conservative prediction to capacity when full credit is given to the benefit of shear reinforcement, particularly for walls subjected to axial compression. Although not statistically significant for such a small sample, it appears that shear capacity predictions are more conservative in cases when flexural tensile stress in the vertical reinforcement are lower (further from yielding) than other cases when shear capacity and moment capacity are more nearly equal. Although not proven here, it is logical that more extensive and wider flexural cracks would result in reduced shear capacity. Therefore, some modifications of shear behaviour should be expected in cases where flexural capacity is reached. This could be especially noticeable for cyclic loading in the range of plastic hinge behaviour.

The observation of partially-filled, poorly compacted grout cells at the ends of block may be thought to have contributed to some weakening of these walls. However, the non-continuous nature of the grout in these cells raises some question regarding the correctness of this assessment. In terms of development of diagonal cracks and flexural cracks, it may well be that the filling or not of these cells is not highly important. The impact on axial load and flexural capacities may be more significant.

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