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SHEAR TESTS ON WIDE-SPACED PARTIALLY REINFORCED SQUAT MASONRY WALLS

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ABSTRACT
In both Canada and Australia, partially grouted concrete block masonry is permitted. The widest distance allowed between reinforcing bars is 2.0 m in the Australian code and 2.4 m in the Canadian. There appears to be no literature describing the in-plane shear behaviour of squat walls with such widely spaced reinforcement. Three 5 m long, by 1.6 m high concrete block walls were subjected to in-plane lateral loading at the top of one end of each wall. The walls were wide spaced reinforced vertically and had a one-course bond beam along their tops. One wall had vertical reinforcement at the ends and at 0.8 m centres, the second at 1.2 m centres and the third with the reinforcement at 1.6 m centres. Thus there were six sections of hollow blockwork in the first wall, four in the second and three in the third. Thus these walls all conformed to reinforcement spacing allowed in the Canadian and Australian masonry codes. With no vertical load, the walls were subjected to in-plane shear loading to failure. The question examined was whether the plain masonry panels (with hollow cores) between the grouted and reinforced cores would act independently or compositely under this loading. The expectation was that the more widely spaced the reinforcement, the greater would be the tendency for independent action of the panels. Wall construction and testing is described, together with the failure modes and strengths observed. Actual strengths are substantially lower than those predicted by code equations.

KEYWORDS: In-plane Shear, Partially grouted masonry, Wide-spaced reinforcement.

INTRODUCTION
Partially reinforced concrete masonry walls are permitted in both the Australian and Canadian design codes [1, 2]. The Australian Standard allows vertical reinforcement in a wall to be spaced up to 2 m apart, whereas the Canadian code allows a maximum spacing of 2.4 m. When the

spacing is greater than 800 mm, the Australian Standard defines the masonry as wide-spaced, partially reinforced masonry. Typically the wide-spaced reinforcement is provided to resist out-of-plane flexure from wind loading. The hollow panels between the reinforced cores have to be able to resist the wind loading through bending and transfer that load to the reinforced cores [3]. Such walls would typically not be built in seismically active areas as the reinforcement is too far apart to provide integrity to the masonry under earthquake loading. In a seismically active zone, the reinforcement needs to be spaced at much shorter intervals. Ingham for example, has used spacings of 800 mm or less for all his testing of concrete masonry for design in New Zealand [e.g.:4]. The Canadian code [1] limits the maximum spacing of reinforcement in seismic zones to 1.2 m. Wide spaced partially reinforced walls will be subject to in-plane shear when they are part of a building and the wind blows on the faces of the building perpendicular to the wide spaced reinforced walls. In contrast to reinforced masonry shear walls where the vertical reinforcement is spaced at 800 mm or less, very little is known about the behaviour of wide-spaced partially reinforced walls when subject to in-plane lateral loading. Some experiments involving in-plane shear have been performed on wide-spaced reinforced walls with an aspect ratio close to 1, and an explicit finite element model has been developed [5-11]. However, no tests to date have examined squat walls consisting of a series of hollow panels between the reinforced cores.

We therefore tested three walls where the overall aspect ratio was 0.32 (1.6 m high by 5 m long). The objective was to determine if there was a shift in failure mode as the spacing of the reinforcement became wider – would walls where the unreinforced panels between the reinforced cores had an individual aspect ratio close to 1 fail as a series of independent unreinforced masonry panels, or would the wall always work as an integrated unit, no matter what the spacing of the reinforcement? The greatest spacing of 1.6 m follows typical practice in Australia, but is short of the maximum 2.4 m allowed in Canada. A wall of the chosen overall dimensions with this latter spacing of the reinforcement would only have had two panels of unreinforced masonry, and it was thought that such a wall would be prone to fail from local end effects rather than providing representative behaviour of a higher wall with the same spacing of the reinforcement.

WALL GEOMETRY AND MATERIALS

The walls were 1.6 m (8 courses) high by 5 m long. One wall had the vertical reinforcing bars spaced at 0.8 m intervals, one with the reinforcing bars at 1.2 m as shown in Figure 1, and one with the reinforcing bars at 1.6 m spacing. The cores containing the reinforcement were grouted. Thus there were six plain masonry segments (panels) in the wall with 0.8 m spacing, four in the 1.2 m spaced wall (Figure 1) and three in the third wall with the 1.6 m spaced reinforcement. In this latter wall, the plain masonry panels were roughly 1.4 m wide (depending on when the hollow masonry is defined to have begun next to the grouted core) by 1.6 m high, thus having an aspect ratio of almost 1.

The walls were constructed on 6 m long base beams, with the vertical reinforcing bars bent into the base beams and spot welded to the stirrups and longitudinal bars (Figure 2). The units were 140 x 190 x 390 mm (W x H x L) hollow concrete blocks. The mortar was a 1:1:6 Portland cement:lime:sand mix by volume. The walls were constructed face-shell bedded by a skilled mason. Three 4-unit high face-shell bedded prisms were tested, giving a strength for the hollow masonry of 12.6 +/- 0.46 MPa, based on net bedded area. The cores with the reinforcement were

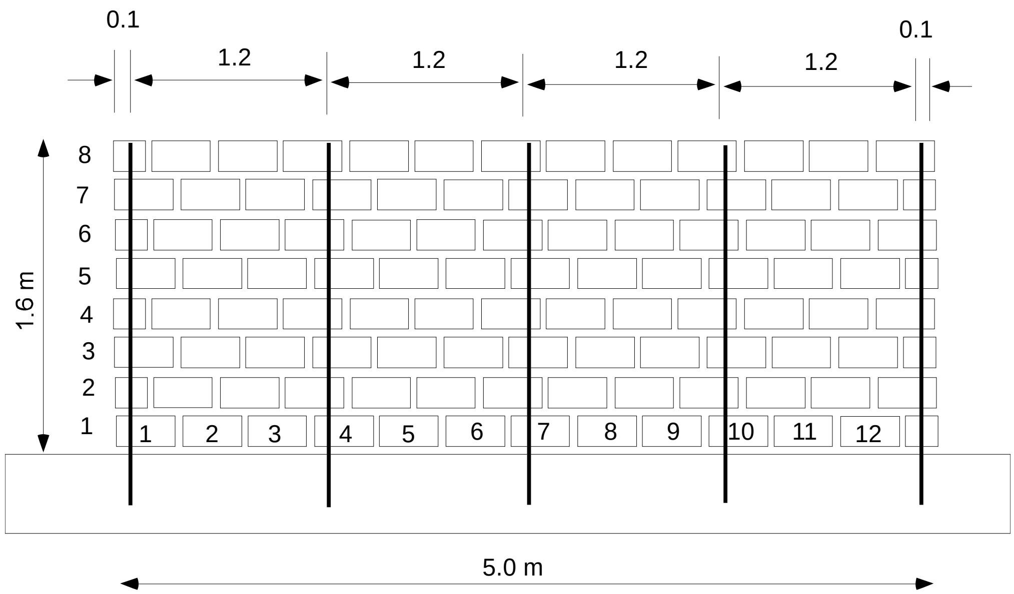

Figure 1: Schematic side elevation of Wall 1 with the vertical reinforcement spaced at 1.2m.

Figure 2: vertical reinforcing bar welded to the beam reinforcement cage.

Figure 3: Grouting of core with reinforcement after four courses had been laid.

grouted after the first four courses had been laid (Figure 3), with the grouting of these cores being completed when the remaining four courses had been finished. The top course consisted of lintel block, in which a reinforcing bar was laid and the grout was poured to form a bond beam. The vertical bars were tied to the horizontal bar in this bond beam. All reinforcement was 12 mm diameter, with a yield strength on test of 535 MPa. Three grout samples were prepared by block moulding as specified in ASTM C1019 [12], giving a compressive strength of 15.5 +/- 3.5 MPa.

INSTRUMENTATION AND TEST PROCEDURE

The walls were instrumented with displacement transducers on each side, mounted diagonally across each hollow panel to monitor for diagonal cracking of each panel. Thus three pairs of transducers were mounted on each side of the wall with the 1.6 m hollow panels, six pairs on the wall with 0.8 m panels and four pairs on the wall with 1.2 m panels, as shown in Figure 4. In addition, transducers were mounted against the vertical faces at the ends of the walls to measure the in-plane deformed shape of the wall. The movement of the concrete base was also monitored with transducers.

Each wall was restrained from out-of-plane movement at three locations along the bond beam on the top of the wall to prevent out-of-plane buckling of the wall, as may be seen in Figure 4. Load was applied to one end of the bond beam through a hand pumped hydraulic actuator (Figure 5). This actuator was braced against the vertical wall of the load floor/frame, with load being transferred through a load cell, a section of HSS and then through a spherical seat to a plate bonded to the masonry (Figure 5). Load was applied until the wall had displaced beyond peak load capacity, and cracked substantially. No vertical load was applied, so the walls were tested under self-weight only.

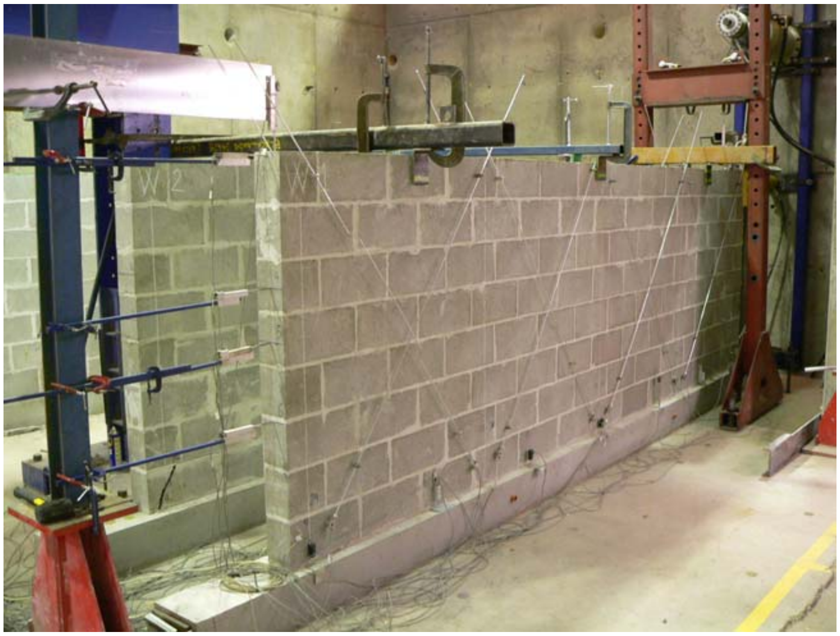

Figure 4: Instrumentation on Wall 1 with four panels of hollow masonry, 1.2m centre to centre. The three bracing points to prevent out-of-plane displacement can be seen at the top of the wall.

Figure 5: The actuator for applying lateral in-plane load at the top of one end of the wall.

RESULTS

The walls deformed in shear as shown in Figure 6 for Wall 1. The displacements measured with the transducers on the end of the wall show linear deformation up the height of the wall. The wall containing 0.8 m spaced vertical reinforcement resisted a maximum lateral load of 220 kN. The crack pattern is shown in Figure 7. It is recognized that if the load had been distributed over the whole top surface of a wall, the cracking patterns and thus failure modes might well have been different to what was seen. The wall with the reinforcing bars spaced at 1.2 m had a peak capacity of 210 kN when tested the second time (the (b) test). In the first test (the (a) test), the stroke of the actuator limited the displacement that could be applied. This wall cracked as shown in Figure 8, while the wall with the reinforcement spaced at 1.6 m reached a maximum load of 160 kN, cracking as shown in Figures 9 and 10. The walls generally failed through an inclined crack emanating from the loading plate position, as can be seen in Figures 7-10. The cracking patterns differ in detail, although there is a tendency for the crack to propagate diagonally across the first plain masonry panel, follow a generally horizontal path and then kink diagonally towards the toe of the wall where local crushing occurred. In the wall with 0.8 m spacing of the reinforcement, the crack followed head and bed joints at roughly 45° as it descended through the panel closest to the point of load application. When the first grouted column was reached, the crack followed a horizontal path through the next three panels of plain masonry before dropping slightly in the next and following a diagonal path to the toe in the final panel. Essentially therefore, there was bed joint sliding failure in the 4 central panels and diagonal failure in the first and last panels. The dominant crack in the wall with reinforcement spaced at 1.2 m followed this pattern although substantial other cracking was observed to give an overall appearance of diagonal cracking. The horizontal cracking can be seen in the two central panels of this wall in Figure 8, but nevertheless, this wall was the one closest to failing with a diagonal crack from the corner of load application to the toe. Cracking in the wall with the 1.6 m spaced reinforcement gave a hint that the wall could have failed as a set of independent panels. During the test, the panel closest to the point of load application, cracked first on its diagonal, as shown in Figure 10. A thin diagonal crack formed in the second panel of plain masonry as well as a horizontal continuation of the crack from the first panel. As the actuator displacement was increased, it was this latter crack that expanded, rather than the second diagonal crack. At the end of testing in all walls, the dominant crack was typically in the order of 10 to 15 mm wide in its vertical segments. The horizontal load versus horizontal displacement plots obtained for each wall are shown in Figure 11. While the walls with 1.2 and 1.6 m spacing of the reinforcement had similar stiffnesses, the wall with 0.8 m spaced reinforcement was considerably stiffer.

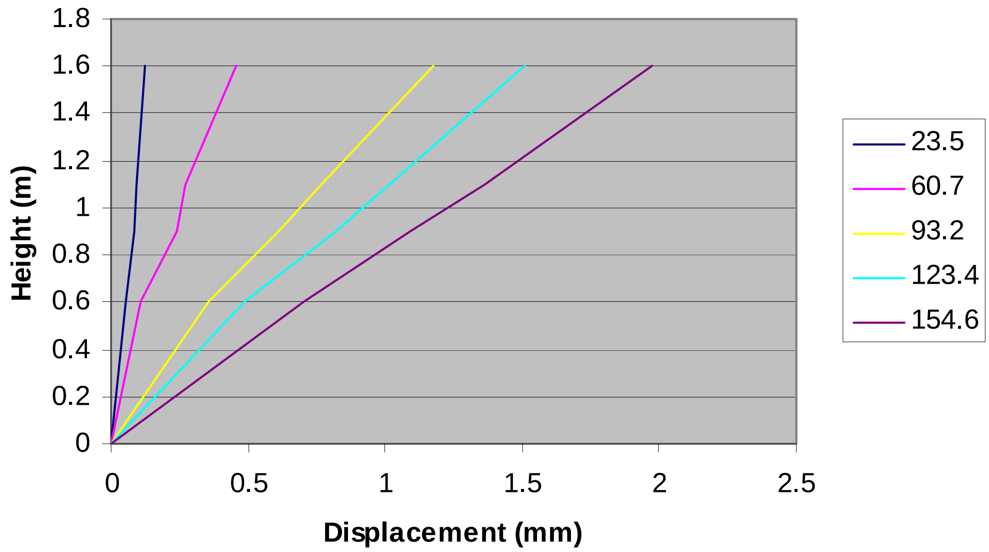

Figure 6: Displacement up the height of Wall 1 (1.2 m spacing) at different loads (kN).

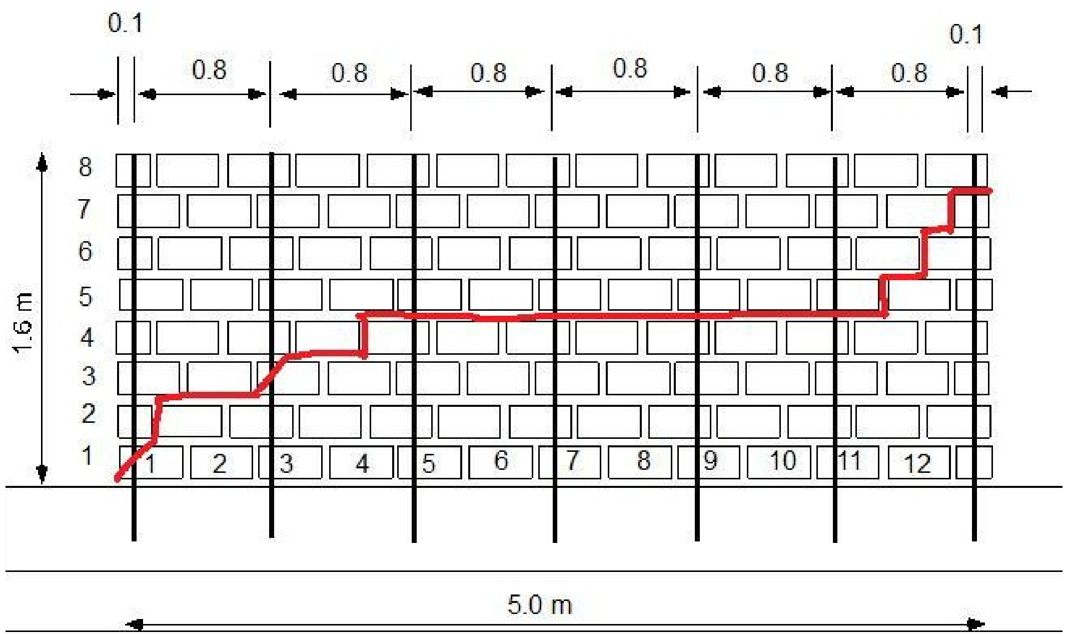

Figure 7: Cracking of wall 3 (0.8 m spaced reinforcement) showed a combination of diagonal and sliding failures. Load was applied at the top right corner in this view.

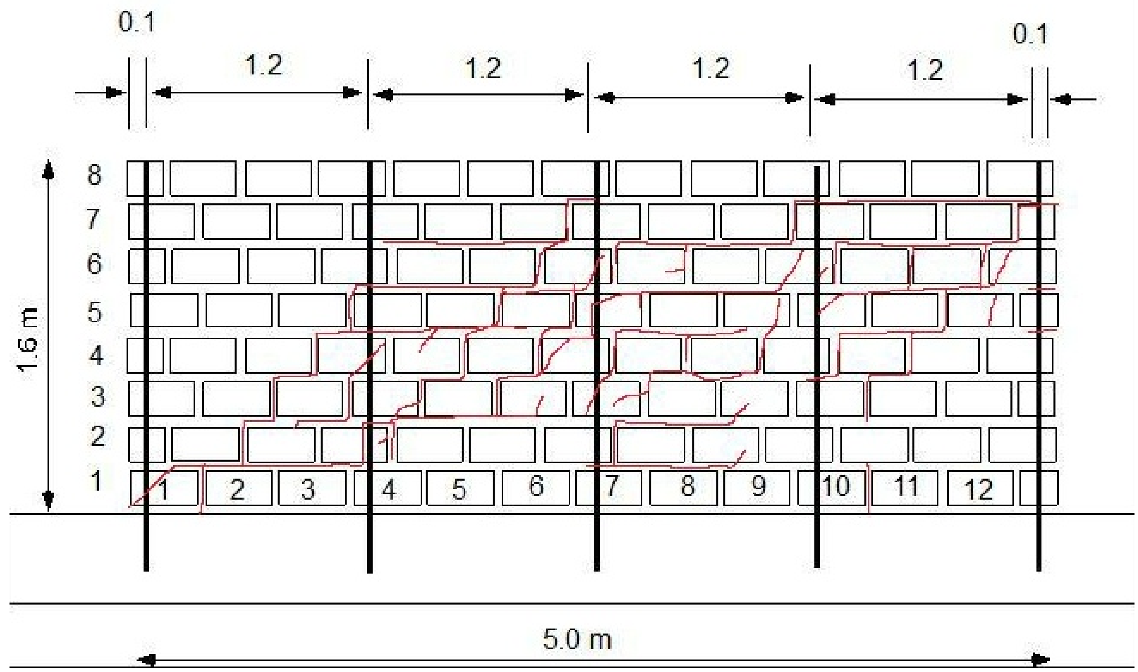

Figure 8: Wall 1 (1.2 m spacing) had many cracks along the diagonal, although the major crack tended to follow a similar path to that in the 0.8 m wall. Load was applied at the top right corner of the wall in this schematic.

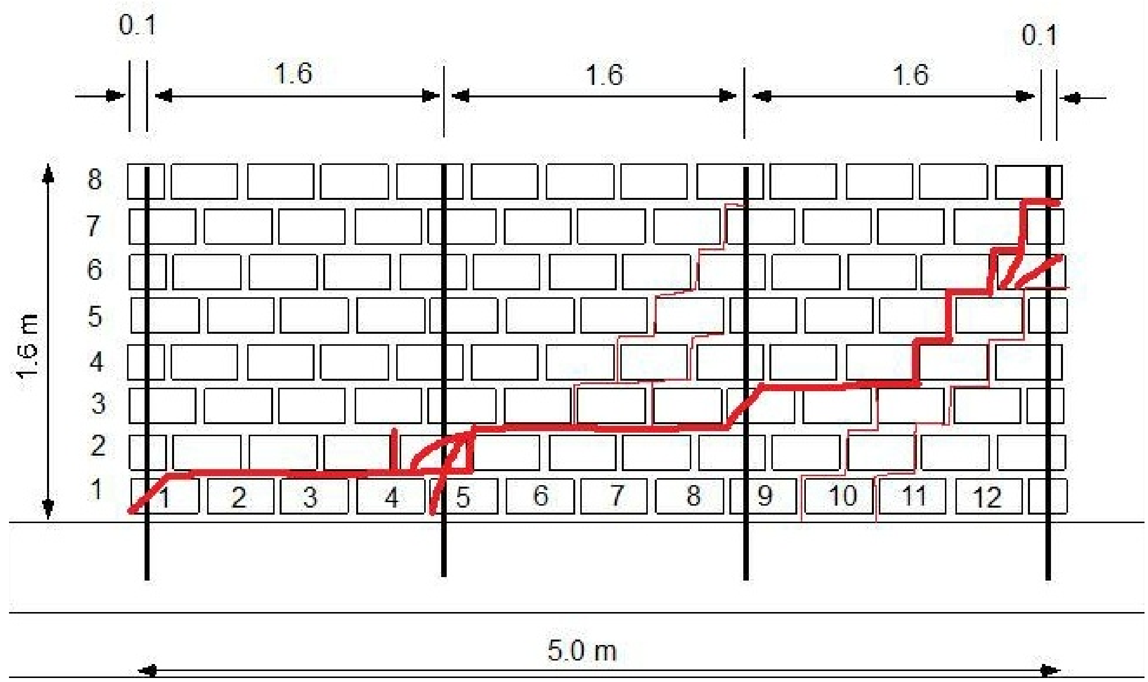

Figure 9: The major crack in wall 2 (1.6 m spacing) ran diagonally from the loading point as in the others and then had a sliding component, although narrower cracks could be seen on the diagonal of the second panel away from the point of load application.

Figure 10: Actual cracking from the loading point (top left of the wall in this view) in wall 2 (1.6 m spaced reinforcement).

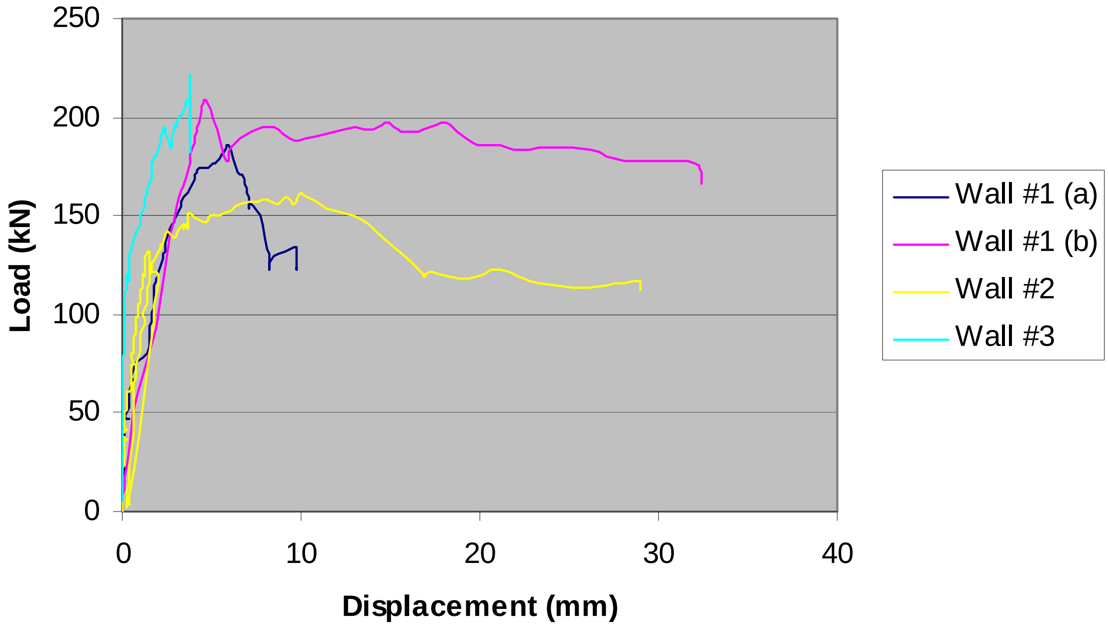

Figure 11: Load-displacement curves for the three walls. Wall 1 (1.2 m spacing) was tested twice as the actuator reached maximum stroke in the 1(a) test and had to be retested. Wall 2 had 1.6 m spacing and Wall 3, 0.8 m.

Table 1: Experimental and code predicted strengths

Specimen Reinforcement Spacing	Experimental Strength (kN)	Strength Calculated from Code (kN)	
		AS3700 [1]	CSA S304.1 [2]
0.8 m	220	760	495
1.2 m	210	702	460
1.6 m	160	625	428

DISCUSSION

The strengths for these walls estimated using the equations for shear strength in AS3700 [1], CSA S304.1 [2] and the respective experimental values are shown in Table 1. These equations were developed from tests on masonry walls and panels where the reinforcement was typically 0.8 m or less, and the walls were also often fully grouted. In these equations, the total shear strength is found by adding the shear resistance of the masonry to the shear resistance of the steel. The shear strength is the minimum of the strengths determined for flexural shear failure, diagonal cracking or bed joint sliding. Combined mechanisms, as are likely to occur in squat walls, are not considered. As the majority of tests from which the equations have been developed have been performed on walls with aspect ratios of about 1, with just a few on slender walls (aspect ratio grater than 1) and even less on squat walls, the effect of aspect ratio is not well understood. However, the Canadian code does have a clause for low aspect ratio (squat) walls: the clause excludes any steel present from contributing to the overall strength, and gives a nominal strength for the walls described here of 371 kN. However, this clause requires the designer to ensure the load is distributed along the top of the wall, rather than having a point of application as in these tests. Here, the bond beam at the top of the wall would have helped distribute the load to some degree, but not as intended by the clause. The estimates from the Canadian code above are for diagonal strength, whereas the mode of failure was a combination of sliding and diagonal failure. The resistances to sliding for the three walls as estimated from the Canadian code, would be 214, 268, and 375 kN. These values are closer to the experimental ones than the estimates for diagonal strength. Importantly, it can be seen that the code equations provide very different estimates of strength, all of which are significantly higher than the strengths actually obtained. It is clear therefore, that the mode of failure of wide spaced reinforced concrete masonry is not well understood, nor do current equations in the codes of practice considered provide good estimates of the in-plane shear strength that may be expected from this type of construction. The difference in the estimates from the codes alone is indicative of the lack of knowledge on this subject. As stated earlier, it is thought that the detailed boundary conditions of the test will affect the failure strength and possibly the failure mode: here for example, the fact that the load was applied as a point load rather than distributed along the top surface of the wall.

The tests described here by no means cover the range of possible geometric and material combinations that could occur in practice. There is a need for the effects of such aspects of this type of construction to be examined. A parametric study using the finite element method, such as that proposed by Haider [9], would be effective, with further experimental tests in regions where changes in behaviour and failure mode are predicted to occur. In the tests here, there was a hint that the panels of plain masonry could act independently in the wall with the 1.6 m spacing of the reinforcement. Indeed, if the load had been distributed along the top surface of the wall, such a response may well have been observed.

CONCLUSIONS

The walls tested had strengths considerably less than predicted by code equations. As such little research has been performed on wide spaced reinforced concrete masonry, it is clear that considerably more work is required so that appropriate clauses can be developed for the safe design of this type of masonry.

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