THE EFFECT OF ASPECT RATIO ON THE SHEAR STRENGTH OF PARTIALLY GROUTED CONCRETE MASONRY WITH WIDE-SPACED REINFORCEMENT

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ABSTRACT

Partially grouted and reinforced hollow concrete masonry is used widely. The maximum spacing allowed between reinforcing bars varies in codes of practice, being for example, 2.0 m in Australia and 2.4 m in Canada. The aspect ratio (height/length) of shear walls has been a component of several studies on the shear capacity of reinforced concrete masonry, with consensus that walls with lower aspect ratios show higher in-plane shear capacity. However, there have been few studies investigating the behaviour of wide-spaced partially reinforced (WSPR) masonry walls. The results of tests on three WSPR concrete masonry walls with an overall aspect ratio of 0.32 but varying spacing of the reinforcement revealed that the shear equations from both Canadian and Australian codes significantly overestimated the shear capacity of the walls. Therefore, to investigate the behaviour of WSPR walls subject to shear further, 8 walls were constructed from half-scale concrete masonry blocks, consisting of 2 square panels of the same size and 6 squat walls of the same height but with an aspect ratio of 0.5. The walls were tested monotonically in in-plane shear. Results showed that two-panel specimens could crack as two independent panels or as a wall as a whole. What causes the difference in behaviour of supposedly similar specimens subject to the same test conditions remains to be elucidated.

KEYWORDS: in-plane shear, partially grouted masonry, wide-spaced reinforcement, aspect ratio

INTRODUCTION

Partially reinforced concrete masonry walls are widely used in modern construction. They are considered as a cost-effective alternative for certain types of building in low and moderate seismic regions [1]. Studies have shown that partially reinforced walls perform well in shear and provide the required ductility for seismic zones [2]. Masonry design codes such as those in Canada and Australia permit partially reinforced concrete masonry in construction [3, 4]. The Canadian code allows vertical reinforcing bars to be placed a maximum of 2.4 m apart (1.2 m in seismically active zones) whereas the Australian code limits the maximum spacing to 2 m. In the Australian code, the wall is considered to have wide-spaced reinforcement when the spacing is greater than 800 mm. The main role of wide-spaced reinforcement is to provide resistance against out-of-plane flexure, from for example, wind load. However, the reinforcement has an effect on the in-plane shear behaviour as well.

Previous studies on the effect of the aspect ratio of the wall on shear strength show that shear capacity increases with decreasing aspect ratio [5-9]. Most of the empirical shear equations in both codes of practice and the literature incorporate the effect of aspect ratio in the estimate of shear strength. However, there is not enough information available to date to determine the relationship between aspect ratio and strength for WSPR. Shrive et al. [10] carried out tests to examine the effect of reinforcement distribution on shear capacity of WSPR. The study revealed that both the Canadian and Australian codes significantly overestimated the shear capacity of the WSPR walls they tested. The objective of this study therefore was to compare the shear strength and crack pattern of single panel square walls with squat walls consisting of two of the same panels. The effect of the initial axial load is also of the interest. The test arrangement was designed to restrain the wall from rotation. Thus the axial load changes during the experiment to as the boundary conditions impose displacement to achieve shear deformation.

WALL GEOMETRY AND MATERIALS

8 walls were constructed from half-size concrete blocks (dimensions of 90 x 90 x 185mm (W x H x L)), two square walls and 6 squat ones. The square walls were 1.23 m high (13 courses) by 1.23 m long whereas the squat ones were 1.23 m high and 2.37 m long. For the square walls 10 mm vertical reinforcing bars were provided in the first and last cores and for the squat ones, three reinforcing bars of the same diameter were spaced 1.14 m apart (one in each of the first and last cores, and one in the middle core). All cores with reinforcing bars were grouted. The top course (of lintel blocks) was also reinforced with a 10 mm steel bar and grouted to form a bond beam. Given the spacing of the reinforcement, the square walls consisted of one masonry segment (panel) while the squat ones had two panels.

The walls were constructed on C-channel steel beams, with the vertical reinforcing bars welded at the required spacing. The walls were built face-shell bedded by a skilled mason using standard type N (1:1:6) mortar. Mortar joints were 5 mm thick to complete the half scale masonry. 3-unit high face-shell bedded prisms were also built during the construction of the walls, and were tested when the corresponding wall specimens were tested. The compressive strength of the prisms was 9.3 +/- 0.51 MPa, based on net bedded area. The cores with reinforcement were grouted every three courses to ensure the required quality of the construction. The grout was a 1:3 Portland cement: sand mix by volume, giving a compressive strength of 10.4 +/- 3.1 MPa.

TEST PRECEDURES

The result of any test depends on how the test has been conducted and the boundary conditions applied to the specimen. In this study, all walls were tested for shear strength using three actuators. The base beam (C-channel) was bolted to the load floor to prevent sliding and a stiff I-beam was placed on top of the wall. The I-beam transferred both the axial and lateral loads to the specimen. The axial load was applied through rollers on the top flange of the I-beam by two actuators and lateral load was applied to the mid-span of the I-beam through a pin connection. The bottom flange of the I-beam was grouted to the bond beam of the wall which transferred the shear by friction. The I-beam distributed the axial and lateral loads over the bond beam.

The test consisted of two steps. First, the two vertical (axial load) actuators were placed in force-control and equal loads were applied to the beam. Then these two actuators were switched to displacement-control and required to hold the vertical displacement constant, thus adjusting the axial load to do so. The lateral load was applied by the third actuator by imposing lateral displacement at a rate of 1 mm/min. Displacement was imposed monotonically, until the load had dropped to 70% of the maximum value attained.

When the lateral load was applied to the specimen, compression increased at the toe end and the load in the vertical actuator near that end decreased to hold the vertical displacement constant, while the load in the actuator at the heel end increased. However, the deformation of the specimen was such that toe-side actuator would have needed to apply tension to keep the wall displacement unchanged at high lateral loads. Since a tension connection was not provided, the specimen and actuator lost contact during the later stages of the test. Nonetheless, the difference between the vertical displacements of each end of the wall remained below 1.5 mm. This displacement implies a maximum rotation of the specimens of 0.03 degrees: the effect of rotation was ignored. Hence, comparison of the results of these tests with those from tests allowing free-rotation of the specimens may not be valid due to the difference in boundary conditions.

In this study, three groups of walls were tested. The first group was the two square, single panel walls subject to an initial applied compressive stress of 0.5 MPa over the face-shell bedded area. The second group consisted of three squat walls (two panels) tested with an initial compressive stress of 0.5 MPa and the last group was the remaining three squat walls subjected to an initial compressive stress of 2.0 MPa (to study the effect of the initial axial on the shear strength).

INSTRUMENTATION

The walls were instrumented with displacement transducers. A diagonal transducer was mounted on each panel to monitor the diagonal cracking of the panel (see Figure 1). Four displacement transducers were placed at one end of the wall to measure the horizontal deflection at different heights. The up-lift of the base and the heel of the wall were monitored as well. Two transducers were placed at the other side to measure the vertical displacement of specimen (Figure 2). Since the top left and right blocks can crack throughout the test and disturb the measurements, the transducers were not mounted directly on the specimen but rather two angles were mounted tightly to the respective blocks and the transducers read the displacements of these mounted angles. The readings from the diagonal transducers are not reliable after the first diagonal crack initiates in the panel: however, the readings can be useful in determining when the major cracks occurred in the specimens. The instrumentation on a squat wall is shown in Figure 1.

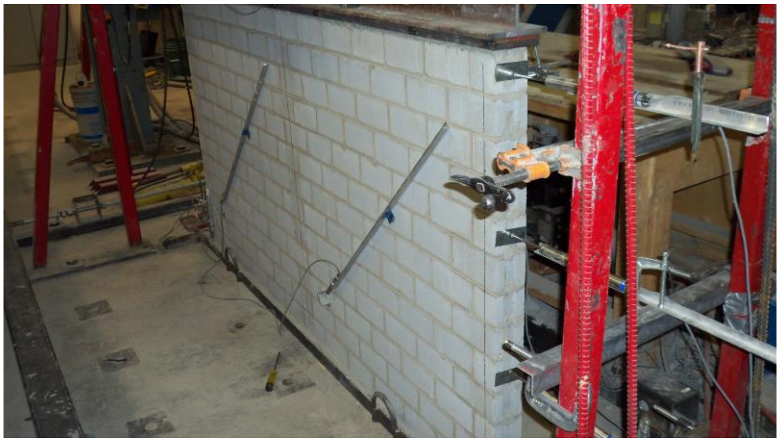

Figure 1: Instrumentation of squat walls

Figure 2. The arrangement of transducers to measure the vertical displacement

RESULTS

A summary of the main test results (maximum shear strength, the corresponding displacement and the axial load) is presented in Table 1. The crack patterns observed in each wall up to peak load are shown schematically in Figures 4, 6, 7, 8, 10, 11, and 12. The numbers in the figures indicate the lateral load at which the crack appeared on the wall. Note that these values do not necessarily reveal the precise load at which a crack initiated, just when it could be seen. However, the loads recorded do show how the cracks initiated and propagated throughout each test. The load-displacement curves for each group are presented in Figures 3, 5, and 9.

Table 1: Summary of results

Specimen	Maximum shear (KN)	Average (KN)	Displacement (mm)	Axial Load (KN)
1a	42.68	43.38	4.52	78.135
1b	44.08		3.67	73.73
2a	110.41	106.60	5.12	190.6
2b	103.7		3.33	190.6

2c	105.68		6.55	196.85
3a	96.22	107.61	5.37	168.02
3b	95.52		5.67	195
3c	131.08		2.7	230.95

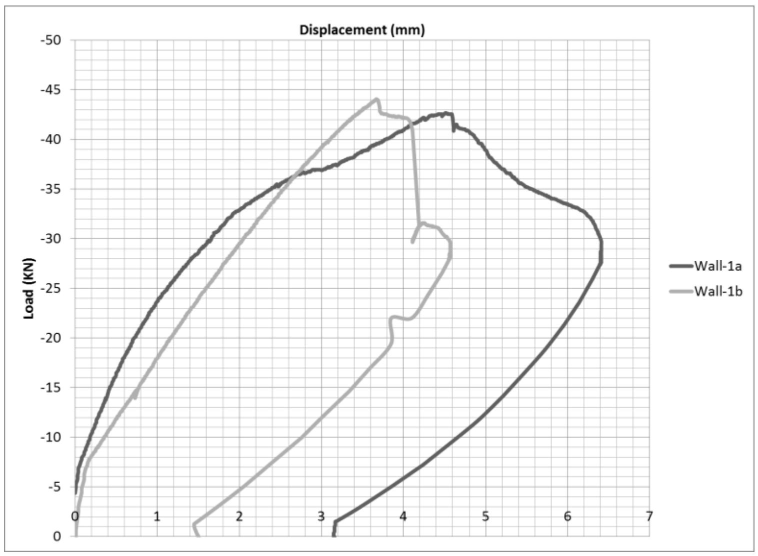

Figure 3: Load-Displacement curves for one-panel walls

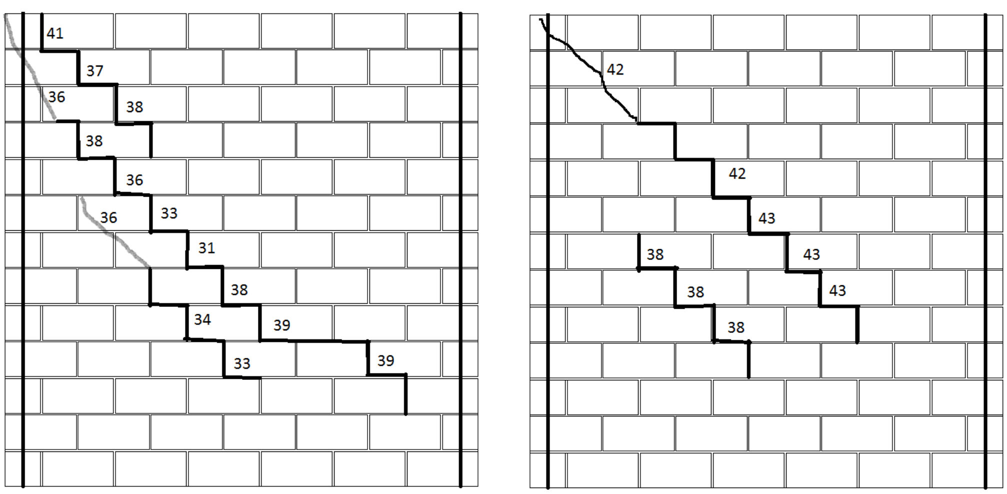

Figure 4: Crack patterns for Wall-1a on the left and Wall-1b on the right

As shown in Figure 3, both square specimens (Wall-1a and 1b) failed in shear, with step-wise cracks initiating in the middle of wall and propagating diagonally. In Wall-1a, the crack appeared early and developed slowly, whereas in Wall-1b, no cracking was observed until the specimen reached close to its ultimate capacity. A diagonal crack appeared on the wall, from the top left corner and extended to the fifth course. The post-failure behaviour of this specimen shows a rapid loss of capacity as shown in Figure 3. The behaviour was brittle. The axial load at the maximum shear strength was higher in Wall-1a even though the shear strength was slightly lower than for Wall-1b. The average shear strength was 43.8 kN for this pair of specimens.

For the second group, the first crack appeared in second panel of Wall-2a (Figure 6). Two major diagonal cracks were observed in the second panel. One developed below the second course from the top of the wall and the other appeared below the third course from the top. The diagonal crack in the first panel did not fully develop but ran along a bed joint and formed a sliding crack. Two separate step-wise cracks appeared in the first panel when the shear load was close to the ultimate capacity of the specimen.

In Wall-2b, two major diagonal cracks were observed in both panels. The load dropped rapidly after the specimen reached its ultimate strength. The displacement at ultimate load was significantly lower than for the other specimens in this group. As can be seen in Figure 7, the two panels acted like two separate panels: no crack passed through the middle core.

The crack pattern of Wall-2c (Figure 8) was similar to that of Wall-2b, but the major cracks from each panel extended to form a sliding crack. The sliding crack in first panel ran through the third course from the top and kinked diagonally towards the sliding crack in the second panel. In the post-failure stage, a major crack developed running diagonally through the specimen as a whole as if there was just one panel in the wall. The average maximum shear strength was 106.6 kN for this set of specimens.

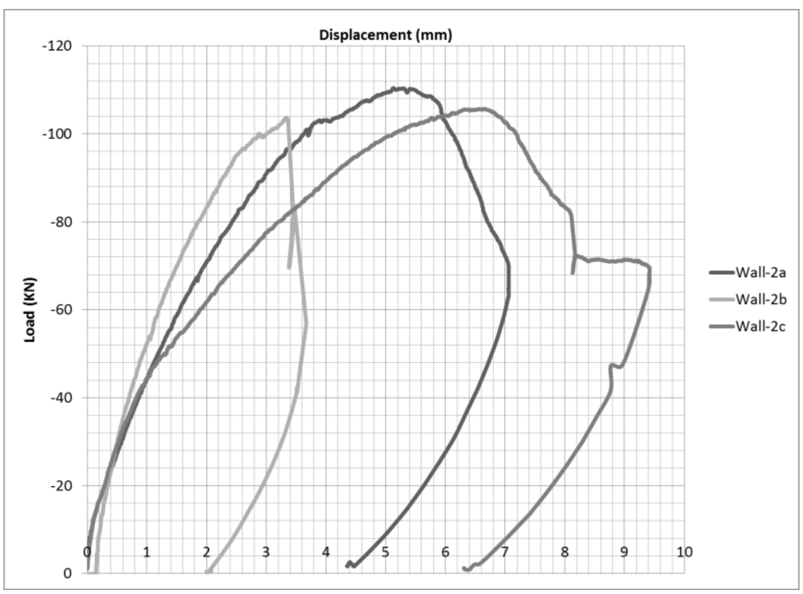

Figure 5: Load-displacement curves for Walls 2a, 2b and 2c

The crack patterns for Walls 3a, b, and c were similar to the ones from the previous group. However, for Wall-3a (Figure 10) and Wall-3b (Figure 11), two maximum loads were detected. There was a small drop in strength in both specimens which was regained and a slightly higher strength was subsequently achieved. The maximum shear strengths of these two specimens were lower than the previous set, despite the higher initial axial load. Wall-3c (Figure 12) showed a different crack pattern from the other squat walls. The crack in the second panel was the extension of the crack from the first panel, which initiated from the top left corner and went through to the toe of the wall – the wall behaving as a single unit than two panels, as in all other cases. The maximum shear strength of this specimen was significantly higher than the other squat walls. The average maximum shear strength of this group was 107 kN which is very close to the second group.

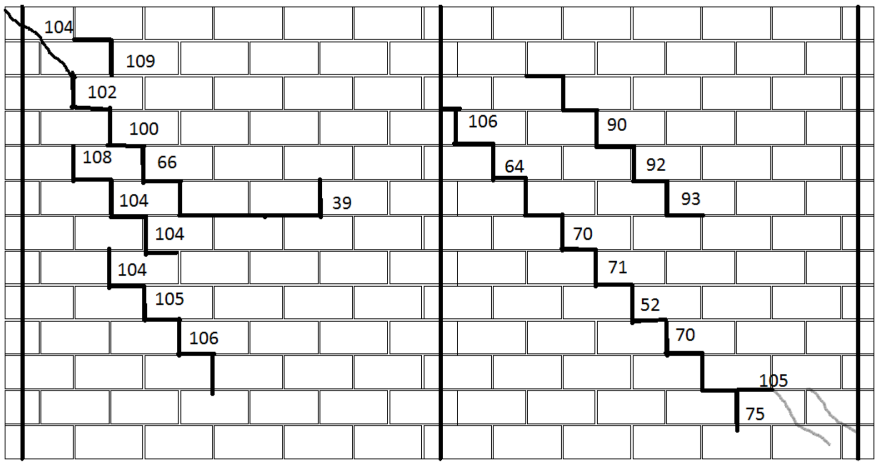

Figure 6: Crack pattern for Wall-2a

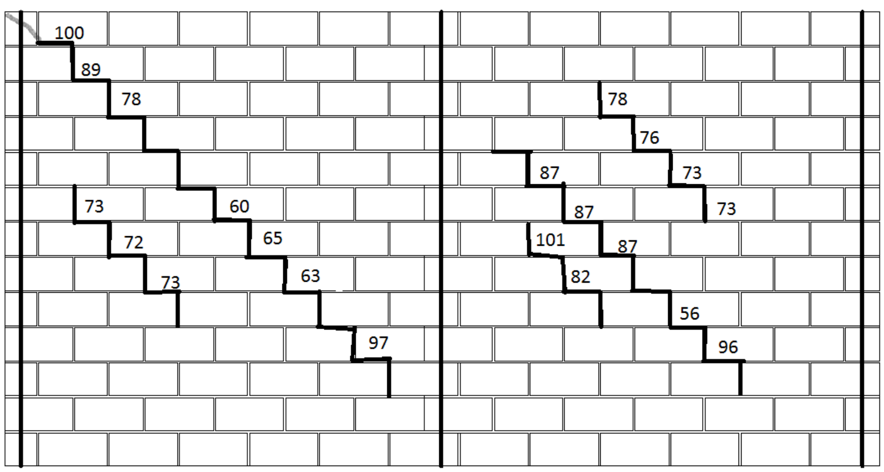

Figure 7: Crack pattern for Wall-2b

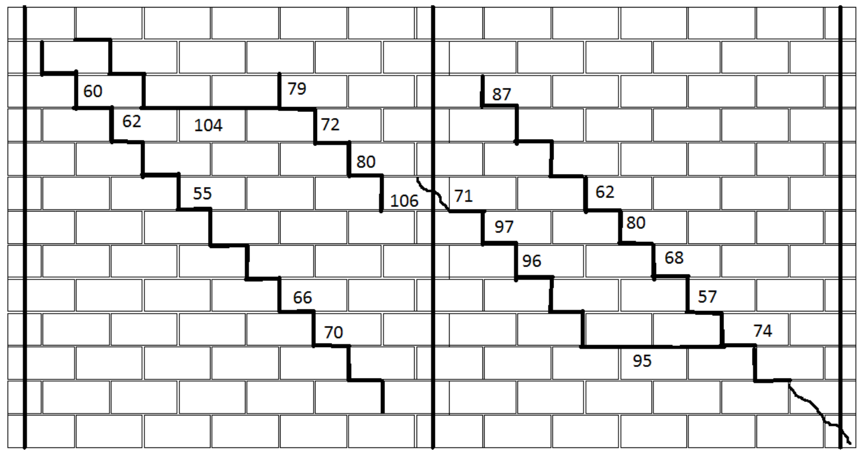

Figure 8: Crack pattern for Wall-2c

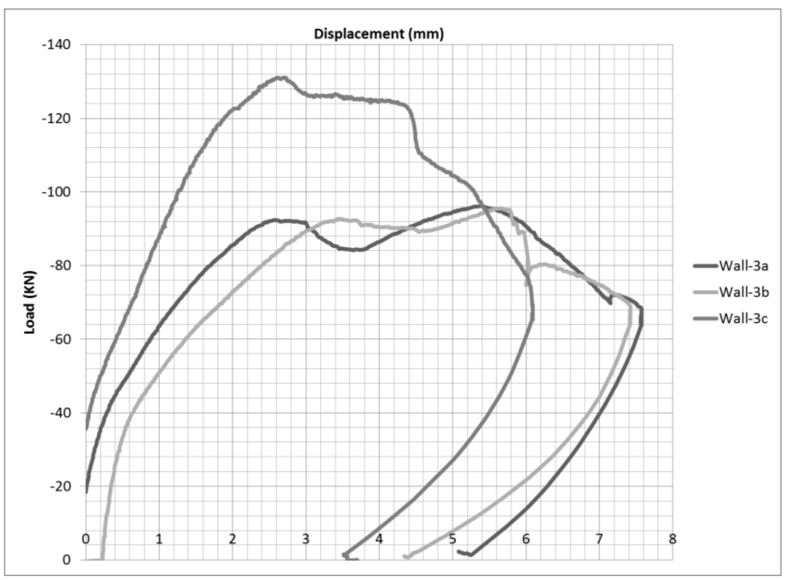

Figure 9: Load-Displacement curves for Walls 3a, b, c

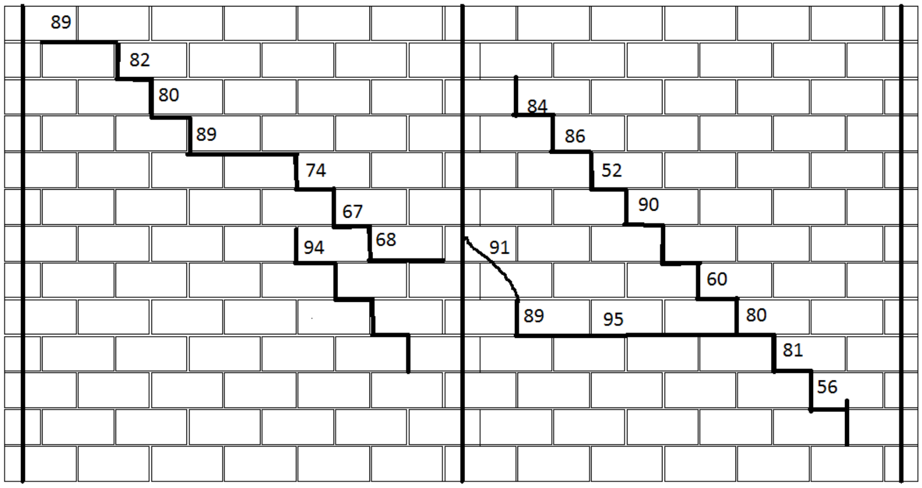

Figure 10: Crack pattern for Wall-3a

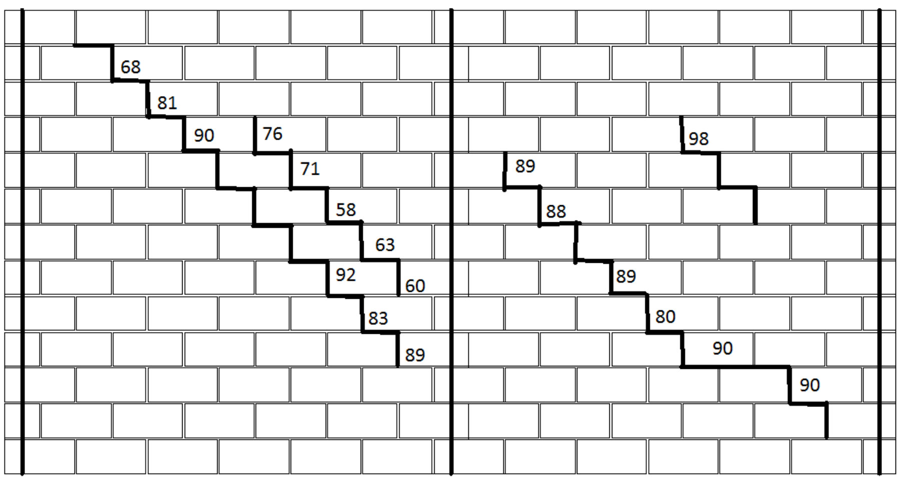

Figure 11: Crack pattern for Wall-3b

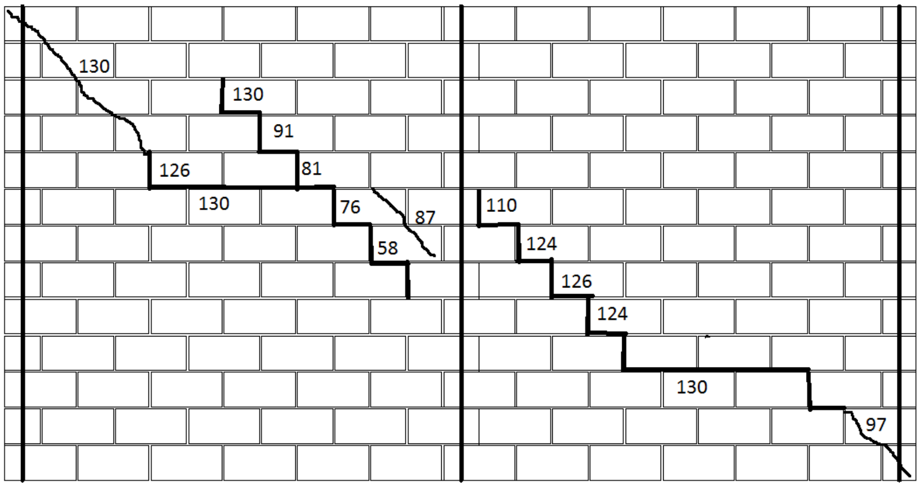

Figure 12: Crack pattern for Wall-3c

DISCUSSION

The main objective of this study was to compare the behaviour of one-panel walls to two-panel walls. The maximum shear strength of walls in group 2 and group 3 was relatively close, but the crack pattern and the progression of failure varied from specimen to specimen. As can be seen, in both groups, there is one specimen where the maximum load and/or displacement is significantly different from the others. For instance, in group 2, Wall-2b (Figure 6) showed very brittle behaviour which resembles unreinforced material in contrast to Wall-2c where the effect of the the presence of reinforcement is obvious. The crack pattern from all the squat walls reveals that brittle behaviour is associated with independent cracking in the first and second panels. As expected, when a sliding crack develops in the wall, crossing or pressing on a vertical bar, the ductile behaviour is more pronounced. The interaction of the panels seems to be the key towards understanding of the behaviour of WSPR. The diagonal crack in the second panel of all squat walls initiated from the second or a lower course from the top. This suggests that the stress transfers from the top two courses, similar to applying a point load to the top corner of the second panel.

No certain conclusion can be made regarding the post-failure behaviour of the specimens. Some showed very brittle behaviour, in contrast to others which showed some ductility. Given the fact that the reinforcement is very widely spaced, this result could have been expected – the wide spacing provides the opportunity for brittle panel behaviour before the steel is activated – but this in turn raises the issue of how brittle vs ductile behaviour can be predicted given the variability in the source materials and construction. The question as to what causes some specimens to be brittle and others to behave in a more ductile fashion remains to be answered. The contribution of reinforcement to overall behaviour of the wall appears to be highly dependent on the initial crack pattern. If the cracks form in such a way that the reinforcement can contribute to the shear resistance, more ductile behaviour is observed: otherwise, WSPR is not very different from unreinforced masonry.

It is of interest to note that the average strength of the two-panel specimens was more than twice the average strength of the one panel specimens, despite the fact that the two-panel specimens had 3 reinforcing bars (1.5 per panel) compared to the two bars in the single panel specimens. Further analysis is required to determine if this is solely an effect of aspect ratio, or if some other factor is also at play. With the higher axial stress, for the two specimens in group 3 which failed in each panel independently, the load was about twice the single panel strength, whereas the specimen which clearly behaved as a complete wall with a single diagonal crack to the toe, had a much higher strength. In group 2, where the failures appeared mainly to be two-panel failures, the loads were more than twice the single panel strength. The results need modelling and further analysis to achieve some understanding.

Lastly we would note that the test method plays an important role. The outcome of this study cannot be applied automatically to other situations where the stress distribution would be different because of the different test parameters and boundary conditions used.

Using half-scale blocks evidently plays an important role in the behaviour of the specimens. The size of the reinforcement bars relative to the area of the grouted cores changes the stiffness of the grouted columns relative to the ungrouted panels between them, which is a key factor in the development of the lateral deformation of the vertical bars, and thus what happens to the panels. Further experimental work needs to be performed to understand this issue more clearly.

CONCLUSIONS

Eight partially grouted walls were tested in in-plane shear. The testing program included three groups of walls: Two one-panel specimens tested at an initial axial compressive stress of 0.5 MPa, three two-panel specimens tested at an initial compressive stress of 0.5 MPa and three two-panel specimens tested at an initial compressive stress of 2 MPa. The results showed the maximum shear strength of two-panel specimen is more than twice the strength of the one-panel ones, in agreement with the findings of the past studies on aspect ratio. The post-failure behaviour of WSPR is not very clear. The crack pattern in the 2-panel walls was either two independent diagonal cracks like two independent panels or a mixture of diagonal and sliding cracks giving the appearance of a compression strut over the whole specimen. What causes the difference in behaviour in supposedly similar specimens subject to the same loading conditions is not clear. The initial compressive stress seemed to have no significant effect on the maximum shear strength of these WSPR specimens, in contrast to other forms of masonry, and this must be investigated further.

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