IN-PLANE CYCLIC LOADING OF PARTIALLY GROUTED MASONRY – A REVIEW AND ASSESSMENT OF RESEARCH NEEDS

N. G. SHRIVE

Killam Memorial Professor
Department of Civil Engineering
University of Calgary,
Calgary, Alberta, Canada, T2N 1N4

A.W. PAGE

Professor Emeritus
School of Engineering
The University of Newcastle
Callaghan, NSW 2308, Australia

SUMMARY

The literature on shear tests on partially grouted masonry is reviewed. Little is known about the effect of shear when the spacing of the vertical reinforcement is greater than 800mm. While the strut and tie approach gives good predictions of load capacity in partially grouted masonry with openings and closely spaced reinforcement, there appears to be no similar capability for wide-spaced, partially grouted masonry. Given the similarity in failure pattern in shear between masonry walls and concrete beams, an equilibrium approach is advocated for examining shear failure in masonry, and tests are suggested to help elucidate basic principles for the analysis of wide-spaced, partially grouted walls subject to in-plane lateral load.

INTRODUCTION

Shear walls in buildings are subject to in-plane shear from wind loading transmitted from a perpendicular exterior wall, or from the effect of seismic motion parallel to the wall. The behaviour of masonry shear walls is therefore of importance to the survival of a building during an earthquake. The behaviour of plain masonry (often called unreinforced masonry (URM)) shear walls subject to in-plane shear and seismic excitation has been studied extensively (e.g. ElGawady et al. 2005), as has fully reinforced masonry (e.g. Shing et al. 1990). However, there is an intermediate form of masonry which has received little attention. Partially reinforced and grouted masonry is typically constructed of clay or concrete blockwork with only the cells containing reinforcement being grouted. The Australian Standard (AS 3700-2001) defines wide-spaced partially grouted masonry as that with the vertical reinforcing bars placed at

intervals greater than 800mm, but not greater than 2000mm. Horizontal reinforcement must be placed at no greater than 3m spacing. The Canadian Code (CSA S304.1-04) allows vertical reinforcing bars to be spaced as widely as 2400mm. Clearly, if the reinforcement is spaced at the wider intervals, with the masonry between being plain blockwork, there is the potential for this form of construction to behave in a manner intermediate between the plain and reinforced cases. The design of wide spaced reinforced masonry is typically governed by flexural capacity – the ability to resist wind loading applied normal to the plane of the wall. However, in recent times there has been a recognition that the behaviour of this type of masonry when subject to in-plane shear is of considerable importance, especially in relation to seismic loading and the establishment of its level of ductility. Recent publications on wide spaced reinforced masonry (Dhanasekar et al. 2001, Dhanasekar and Haider 2004, Haider and Dhanasekar 2004a, 2004b) confirm the paucity of information on this form of construction.

There are three independent modes of failure for masonry shear walls – sliding along a bed (or slip) joint, diagonal cracking (either stepped through head and bed joints, or angled through unit and mortar) or rocking (Figure 1). Rocking (flexural) failure can take the form of tensile cracking near the heel of the wall, crushing of the masonry at the toe of the wall, or a combination of these two. While the major modes are independent, combinations have also been reported. For example, Benli and Houqin (1991) describe tests on walls with an aspect ratio (height/length) of 0.65. The walls had belts of horizontal reinforcement at one third and two thirds of the height. The walls were subject to vertical compression and cyclic reversed in-plane lateral loading. The cracking pattern was observed to be either roughly along both diagonals over the whole wall, or a diagonal/horizontal sliding pattern within the masonry between the lower reinforcement belt and the base, or between reinforcement belts. The horizontal crack in these cases was at the mid height of that section of masonry. These sections had aspect ratios in the order of 0.22. The shear strength was observed to be higher when

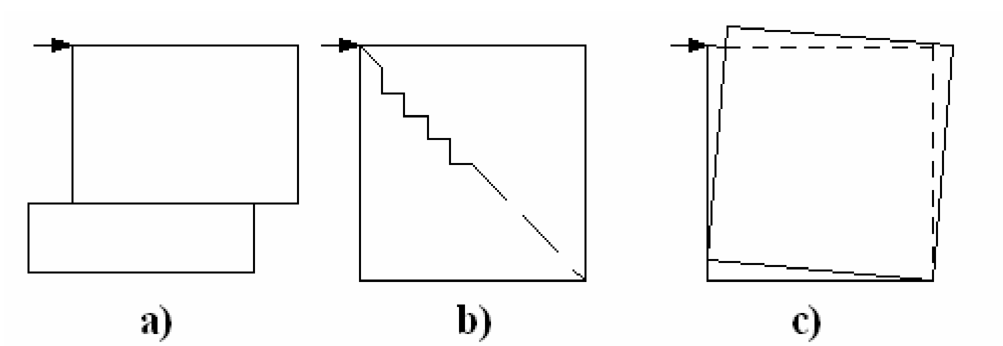

Figure 1. The three modes of failure – sliding, diagonal cracking (stepped through head and bed joints, or through units and mortar) and rocking (heel lifting, toe crushing or both)

failure occurred along a horizontal joint, as opposed to all diagonal failure. The authors also noted the higher the mortar strength and the axial load, the higher the difference in the two modes (diagonal in the whole wall vs. diagonal sliding in one section). Shing et al. (1990) showed that the failure mode of fully reinforced masonry shifted from flexure to shear by increasing the level of axial stress. The shift was accompanied by a reduction in ductility.

FACTORS AFFECTING MODE OF FAILURE

A range of factors can potentially influence the mode of failure of partially reinforced and grouted walls.

Level of Axial Load

Axial stress has been shown to be a major factor in respect of the failure mode. “Slender” walls (ones with an aspect ratio higher than 1) are prone to fail in flexure, often through tensile cracking at the heel and lifting of that part of the wall. As increasing axial compressive stress is applied, simulating the effect of increasing dead load, so the tensile cracking will be suppressed. If there is sufficient compressive strength capacity at the toe to resist both the applied axial stress and the compression induced by the bending, then failure will occur through the next “weakest link” – cracking along the diagonal. Thus increasing the axial load can shift the mode of failure from rocking to diagonal cracking. Diagonal cracks are typically more or less parallel to principal compression, perpendicular to principal tension, and are often called “shear” cracks. The direction of these cracks is not just governed by the local principal stresses, but also by energy demands. Cracks follow the path requiring least energy, which can alter the crack angle away from the principal direction (Loov 2000, Loov and El Metwally, 2002).

Aspect Ratio

Other factors known to affect the strength of a wall in respect of in-plane shear are the aspect ratio, the strength of the masonry, and for reinforced masonry, the type and quantity of reinforcement (e.g. Jianguo and Jingqian 1986, Steelman & Abrams, 2007). Most laboratory tests have been performed on walls and panels with aspect ratios close to 1. Walls with aspect ratios greater than 1 tend to fail through rocking when subjected to cyclic loading, with the development of a “plastic hinge” at the location of failure in reinforced walls. There is much less information available on walls with aspect ratios less than one. Voon et al. (2000) report on the testing of 12 nominally reinforced cantilevered concrete masonry walls, of which 9 were partially grouted. The walls were 2.4 m high and of different lengths (800 to 4200 mm) and widths. Three of the 90mm walls were solid grouted and only one partially. All 140 and 190 walls were partially grouted and had stirrups in the bond beams at the top of the walls. Vertical steel was lap-spliced to starter bars coming out of the concrete base, spaced at 800mm, except for the two 800mm walls where the vertical rebar was 600 mm apart. The only horizontal steel was in the bond beam (top and bottom thereof). Horizontal cyclic loading was applied. All partially grouted walls failed in “diagonal tension” except the 140 wide 800mm wall (hinge sliding plus diagonal tension). The largest ductility was for walls with an aspect ratio of about 1, the ductility being reduced for both shorter (800mm) and longer (4200) walls. The authors argue that this was because of the rapid development of wide cracks which contributed to shear displacement, accelerated initiation of the diagonal tension mode and created strength degradation.

Schultz et al. (2000) describe tests on six walls 1422 mm high, but of different lengths to give aspect ratios of 0.5, 0.7 and 1.0. Welded wire grids (ladder reinforcement) were embedded in

every other mortar joint. The walls had vertical rebars in the outer cells only. When cyclic racking was performed, damage initiated with the formation of vertical cracks in the top course near both jambs, at the interface between grouted vertical cells and adjacent ungrouted masonry. The bed joint reinforcement appeared to bridge vertical cracks and prevent interruption of stress flow. No wall failed suddenly. Multiple fractures of the bed joint reinforcement were observed in all walls when large displacements were applied. Bed joint reinforcement was seen to be an effective form of horizontal reinforcement. Zhuge and Mills (2000) tested walls 6 blocks long by 7 courses high (aspect ratio of 0.58), showing that reinforcement increased ductility considerably. The authors indicate that a spacing of 1000 mm for the vertical reinforcement gave results little different to a spacing of 1800 mm. However, given the size of the specimens, only one space of the latter width could be accommodated in a specimen of the size tested. Neither these authors nor El Gawady et al. (2005) discuss the effect of aspect ratio explicitly. However, for the latter, the square walls failed by rocking whereas the squat walls failed in a mixed rocking/shear mode. For the walls with the stronger mortar, the square wall failed at about the same lateral load as the squat wall with the same mortar. However, for the weaker mortar specimens, the squat wall failed at the same load as its strong mortar counterpart, but the square wall failed at about half the load of its strong mortar companion. The squat walls of Schultz et al. (2000) were some 50% and 15% stronger than their square equivalents for horizontal reinforcement ratios of 0.00056 and 0.0011 respectively.

Influence of Reinforcement

The effect of reinforcement is substantial, increasing both strength and ductility. The steel should be well distributed and the horizontal and vertical steel should have compatible (similar) yield stress to provide good ductility (Khattab and Drysdale, 1993). Increasing the quantity of steel can shift the failure from flexure to diagonal cracking. Some authors suggest from their test results that there is a threshold in the reinforcement ratio, beyond which there is little gain (e.g. Scrivener (1969)) whereas others did not find the same effect (Khattab and Drysdale, 1993). Most of the tests on reinforced masonry have involved full grouting of the specimens. However a limited number of tests have been performed where only the reinforced cells have been grouted. While the masonry is thus partially grouted, the spacing of vertical reinforcement has typically been 800mm or less (Scrivener 1969, Voon et al. 2000, Ingham et al. 2001, Voon and Ingham 2004, 2005). Voon et al. (2000) suggest that the shear resistance of partially grouted masonry consists of frame action generated by the bond beam, dowel action of the vertical steel and sliding friction. This is a little different to fully grouted and reinforced masonry, where the shear strength appears to be governed by aggregate interlocking, dowel action of reinforcement, truss action of flexural and shear reinforcement, and shear resistance at the compression toe. This is a little different to codes where there are typically two components to ultimate strength – that of the masonry and that of the steel. The well accepted influence of vertical load may be included in the masonry shear strength. For partially grouted masonry with openings, strut and tie analyses were found to be better predictors of strength than estimating frame action caused by the reinforced bond beam working over the various sections (Voon and Ingham 2005). Partially grouted walls with openings can develop considerable inelastic displacement capacity, with gradual degradation of capacity after the peak has been passed.

Wide Spaced Reinforcement

The behaviour of partially grouted masonry with more widely spaced reinforcement subject to in-plane shear has received little attention. The tests that have been performed (Dhanasekar et al. 2001, Dhanasekar and Haider 2004, Haider and Dhanasekar 2004a, 2004b) have been on almost square specimens which failed through diagonal cracking under reverse cyclic loading.

The dimensions were such that only one “panel” of unreinforced masonry could be included for the widest core spacing tested. A finite element model was developed to assess the potential effects of parameters not investigated experimentally (Haider 2007). This modeling revealed a potential for different cracking patterns to occur in different specimens. For example, two walls with an aspect ratio of 0.5 were modeled and tested (2870 long by 1411 high). The reinforcement was spaced symmetrically about the mid-line of the wall, with a central 780 mm spacing between outer 960 mm spacings. The FE model suggested that the three panels of plain masonry might each crack along their diagonals, or by diagonal cracks running through one panel and partially into the next. Cracking along the diagonal of the complete wall was not predicted. In the experiments the walls failed by diagonal cracks starting in each of the two outer panels, joined by a horizontal crack right across the centre two reinforced cores and the 780 mm panel – reminiscent of the failures observed by Benli and Houqin (1991).

RESEARCH NEEDS

The literature shows that while considerable progress has been made on understanding and predicting the behaviour of partially grouted masonry with the reinforcement spacing at 800 mm or less, there is much not known about wide spaced partially grouted masonry. Does wide spaced, partially grouted masonry ever fail through strut action in the unreinforced panels between the reinforced cores, and if so, under what conditions? Certainly this type of failure appears to be a possibility (Haider 2007). When does the failure of this type of masonry simply become sections of plain masonry between individual reinforced cores? Some possibilities are shown in Figure 2. The dotted lines are indicative of the locations of reinforced cores. The panels between the reinforced cores may act as such and fail with diagonal cracking in each panel, as shown at the top. This is almost the beginning of mixed behaviour, where the panels act as unreinforced masonry between the reinforced cores. Another possibility is that a diagonal crack forms across the leading one or two panels and then the crack progresses along the base of the wall from either flexural action (as suggested by Brunner and Shing (1996)) or sliding. Alternatively, the sliding could occur at mid height as shown by Benli and Houqin (1991), and shown in the third sketch. Finally, a crack could propagate diagonally through the length of the wall, as shown schematically at the bottom of Figure 2.

The work to date shows that at a spacing of 800mm or less, there is integrated behaviour between the masonry and the reinforcement. Full and partial grouting produce nearly the same strength on a net stress basis (Voon and Ingham 2004). For the almost square specimens tested by Haider and Dhanasekar (2004b), the cracking was more or less on the diagonal of the wall –

failure perhaps being dictated by the specimen shape. When the aspect ratio was reduced, a different crack pattern was noted, as above. Thus it is clear that tests are needed on specimens where the spacing is “wide”, and more than one wide space is accommodated in the specimen. Different wide spacings are required to determine the effect of spacing on the crack pattern. As there are so many parameters which can affect the failure mode and pattern, FE analysis is also required to determine which factors are the most important for this type of masonry, and the likely effects. The important features predicted by the modeling must then be verified by experiment.

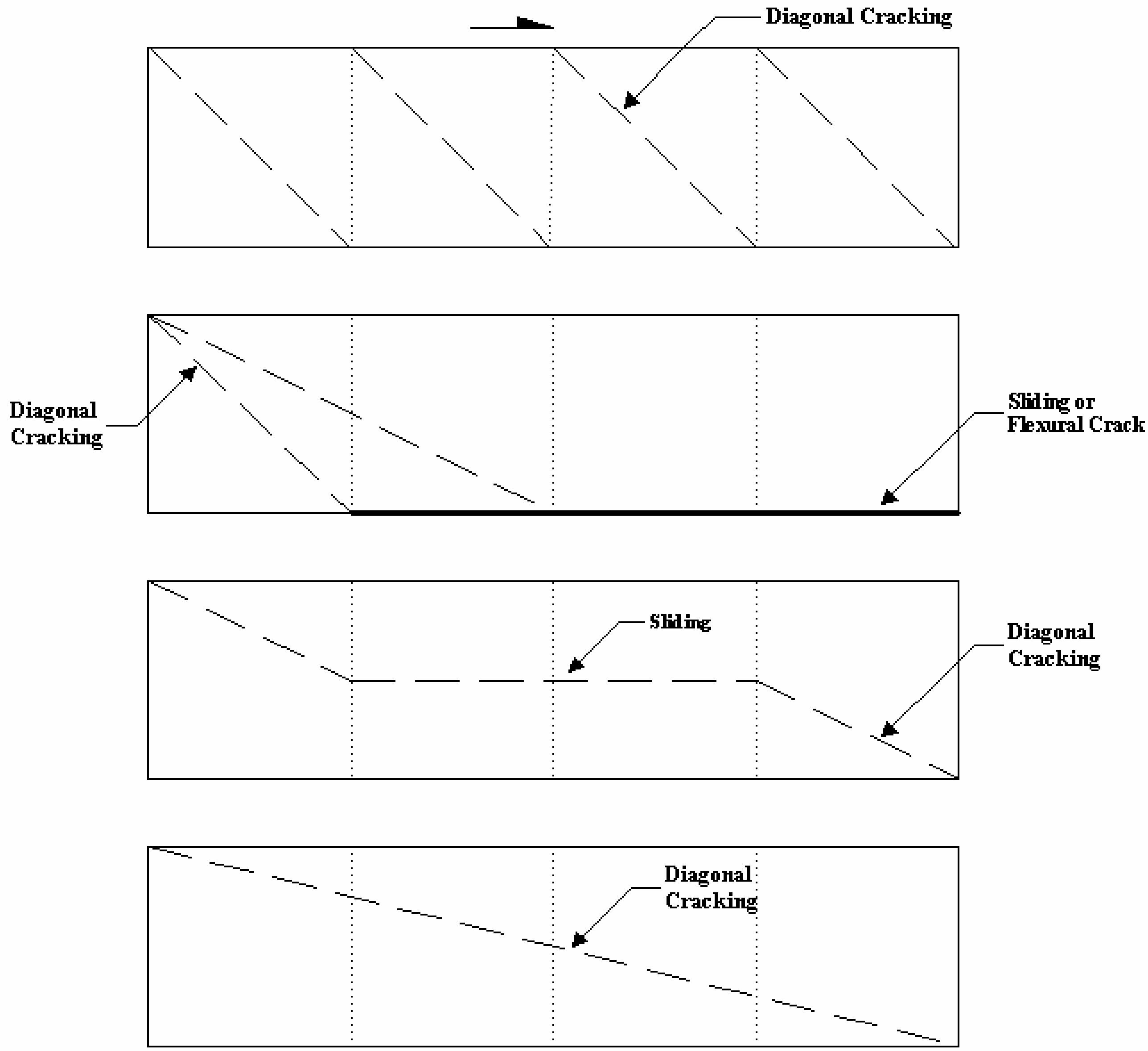

Figure 2. Possible crack patterns for wide-spaced partially grouted masonry.

As highly specialized non-linear finite element modeling is not going to be implemented in a design office on a routine basis, a more straightforward method of analysis and design needs to

be developed – one that lends itself to codification. Codes of practice have had highly differing methods to determine masonry shear strength (Anderson and Priestley 1992), and continue to do so. Methods of determining shear strength of masonry elements also produce widely varying capacities. The variation reflects the lack of confidence that any single method is correct in modeling what is actually occurring. The same problem exists in the design of concrete beams for shear – "The shear design of concrete beams has been hampered by the lack of a clear understanding of the shear support mechanism" (Loov 2000). In Figure 3 we reproduce a mirror image of Figure 1 of Loov (2000) and Loov and El Metwally (2002), rotated through 90°, and compare it with Figure 8a of Bernadini et al. (1997). The similarity is obvious, despite the fact that Bernadini et al. (1997) do not show a bend in their horizontal reinforcement (equivalent to the stirrups in the beam). The approach taken by Loov is one of equilibrium, and it produces continuous and logical results.

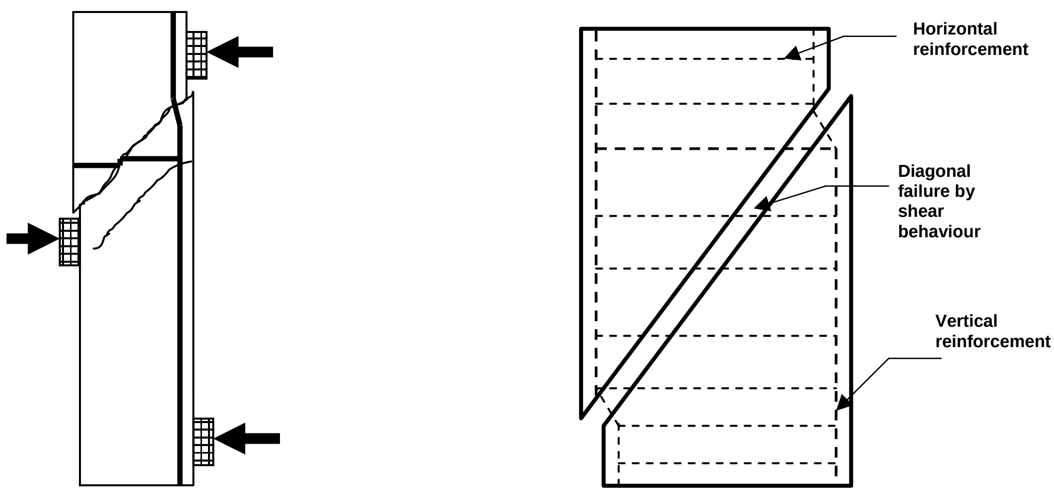

Figure 3. A mirror image of Figure 1of Loov (2000) and Loov and El Metwally (2002) rotated through 90° on the left, and a reproduction on Figure 8a of Bernadini et al. (1997) on the right.

An equilibrium approach is one in which a free body diagram of one part of the cracked wall or beam is drawn. The section chosen to create the free body diagram is the crack itself, as shown in Figure 4, the section on the right being that portion of the masonry above and to the left of the crack in the sketch on the left. Equilibrium and geometric constraints are used to determine the unknowns. Tensile forces in reinforcement, for example can be taken as the yield stress times the total cross-sectional area of the reinforcement. The compressive force in the masonry could be taken as a stress block, depending on the type of failure. F and C could be horizontal and vertical respectively in stepped cracking, or normal and parallel to the crack in angled cracking through the units. The lines of action of these forces will depend on the level of axial load and the mode of failure.

Brunner and Shing (1996) developed a method of analysis based on an equilibrium approach. The method showed a remarkable ability to predict the strength of the 22 walls that Shing and various coworkers had tested in previous years. These walls were all square, and the analytic technique was not applied to the results reported by other researchers. The method involved iteration to find an appropriate neutral axis for the bending component of the analysis, and

made the assumption that all diagonal cracks would occur at 45° to the horizontal. This may be reasonable for walls of roughly square elevation, but is not applicable to walls of other aspect ratios. Also, for wide spaced partially grouted masonry, there is a clear possibility of cracks in the unreinforced panels to be at angles other than 45°. Nevertheless, given the success of the equilibrium approach to both shear in concrete beams and in its limited application to shear of masonry walls, it would appear worthwhile to re-examine this approach. Equilibrium is a natural concept for structural engineers, so a method needs to be developed which can be followed easily. Beginning with wide spaced reinforcement, such a method should be able to predict results obtained for plain masonry and be extended to include fully reinforced masonry. In such a way, a consistent method for the design of solid walls might be developed. Whether such a method could also be applied to walls with openings is moot, as the strut and tie model appears to work well for such walls (Voon and Ingham 2005).

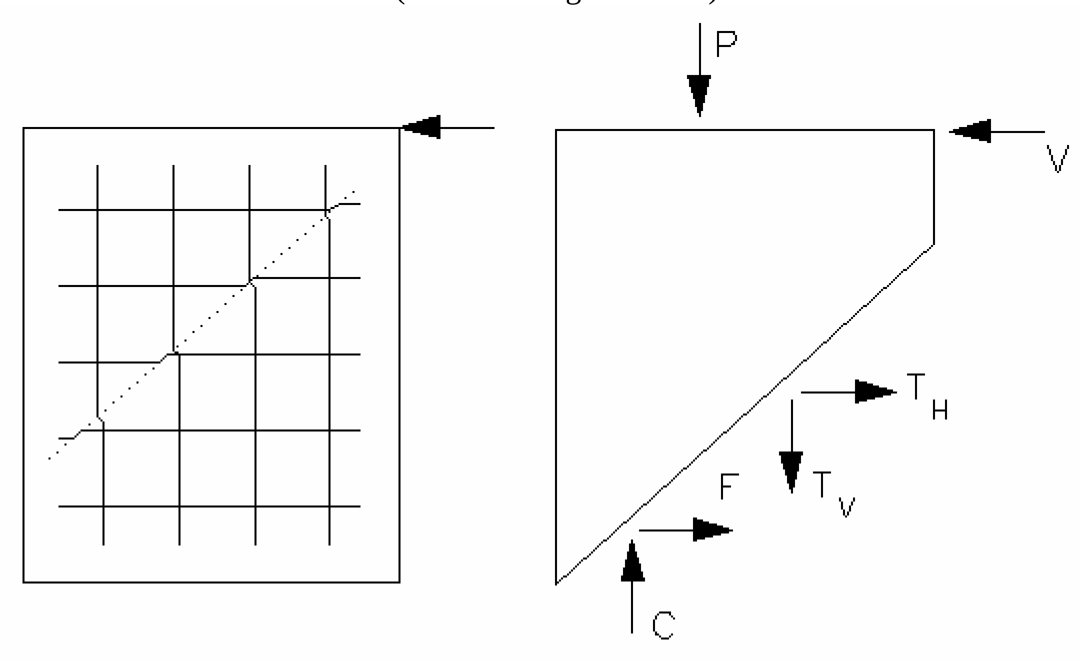

Figure 4. Crack pattern and Free Body Diagram for the equilibrium approach.

Based on this assessment, tests will be performed on walls with multiple wide spaced reinforced cores, the equilibrium approach will be examined to see if a methodology can be developed for masonry shear walls that predicts cracking patterns and strength consistently and with acceptable accuracy for codification. Lastly, finite element modeling will be undertaken to expand the range of variables examined, with follow-up tests on the most critical factors.

CONCLUSIONS

The little work that has been performed on wide spaced partially grouted masonry subject to in-plane shear has revealed how little is actually known about the performance of this type of masonry. Examination of other work on the shear strength of masonry walls shows that there

are numerous factors which affect wall strength, and all of these need to be examined in the context of wide spaced partially grouted masonry. A method of analysis needs to be developed, based on equilibrium principles much along the lines proposed for reinforced concrete beams.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the support of Think Brick Australia, the Concrete Masonry Association of Australia, the University of Newcastle and the University of Calgary.

REFERENCES

Anderson, D.L., Priestley, M.J.N., “In Plane Shear Strength of Masonry Walls”, Proceedings, 6th Canadian Masonry Symposium, Saskatoon, 1992, pp. 223-233.

AS 3700-2001, “Masonry Structures”, Standards Australia International, Sydney, NSW, Reissued containing Amendments 1 and 2, 2003.

Benli, G., Houqin, Z., “The Seismic Resistance Behavior of Partially Reinforced Brick Walls”, Proceedings, 9th International Brick/Block Masonry Conference, Berlin, 1991, pp. 481-489.

Bernardini, A., Modena, C., Lazzaro, G., Valluzzi, M.R., “Cyclic Behaviour and Modelling of Reinforced Masonry Panels”, Proceedings, 11th International Brick/Block Masonry Conference, Supplementary Volume, 1997, pp. 40-49.

Brunner, J.D., Shing, P.B., “Shear Strength of Reinforced Masonry Walls”, The Masonry Society Journal, Vol. 14, No. 1, 1996, pp. 65-77.

CSA S304.1-04, “Design of Masonry Structures”, Canadian Standards Association, Mississauga, Ontario, 2004.

Dhanasekar, M., Steedman, M., Nguyen, H., “Effect of Spacing of Reinforcement on the In-Plane Shear Response of Masonry Walls”, Proceedings, 6th Australasian Masonry Conference, Adelaide, 2001, pp. 135-144.

Dhanasekar, M., Haider, W., “Behaviour of Wide Spaced Reinforced Masonry Walls Under In-Plane Cyclic Loading”, Proc., 13th Int. Brick/Block Masonry Conf., 2004, pp. 535-544.

ElGawady, M.A., Lestuzzi, P., Badoux, M., “Performance of Masonry Walls Under In-Plane Seismic Loading”, The Masonry Society Journal, Vol. 23, 2005, pp. 85-104.

Haider, W., PhD Thesis, Central Queensland University, Rockhampton, Australia, 2007

Haider, W., Dhanasekar, M., “Experimental study of monotonically loaded wide spaced reinforced masonry shear walls”, Australian J. Struct. Engg., Vol. 5, No. 2, 2004a, pp. 101-118.

Haider, W., Dhanasekar, M., “Deformation Characteristics of Wide Spaced Reinforced Masonry Shear Walls”, Proc., 7th Australasian Masonry Conf., Newcastle, 2004b, pp. 93-101.

Ingham, J.C., Davidson, B.J., Brammer, D.R., Voon, K.C., “Testing and Codification of Partially Grout-filled Nominally-reinforced Concrete Masonry Subject to In-Plane Cyclic Loads”, The Masonry Society Journal, Vol. 19, No.1, 2001, pp. 83-96.

Jianguo, F., Jingqian, X., “Experiment Study of Shear Strength of Reinforced Masonry Walls Under Cyclic Loading”, Proc., 4th Canadian Mas. Symposium, Fredericton, 1986, pp. 581-595.

Khattab, M.M., Drysdale, R.G., “The Effect of Reinforcement on the Shear Response of grouted Concrete Masonry”, The Masonry Society Journal, Vol. 12, No. 1, 1993, pp.38-44.

Loov, R.E., “Shear Design of Concrete – A Simpler Way”, Proceedings, 3rd Structural Specialty Conf., Canadian Society for Civil Engineering, London, Ontario, 2000, pp.49-56.

Loov, R.E., El Metwally, A., “The Shear Design of Reinforced and Prestressed Beams”, Proceedings, FIB Conference, Osaka, 2002, paper W-102.

Schultz, A.E., Hutchinson, R.S., Cheok, G.C., “Seismic Performance of Masonry Walls with Bed Joint Reinforcement”, Proc., 12th Int. Brick/Block Masonry Conf., 2000, pp. 1625-1631.

Scrivener, J.C., “Static Racking Tests on Concrete Masonry Walls”, Designing, Engineering and Constructing with Masonry Products, Proceedings, First International Conference on Masonry Structural Systems, Houston, Gulf Publishing, 1969, pp.185-191.

Shing, P.B., Schuller, M., Hoskere, V.S., “In-Plane Resistance of Reinforced Masonry Shear Walls”, Journal of Structural Engineering, ASCE, Vol. 116, No. 3, 1990, pp. 619-640.

Steelman, J., Abrams, D.P., “Effect of Axial Stress and Aspect Ratio on Lateral Strength of URM Shear Walls”, Proc., 10th North American Masonry Conf., St. Louis, 2007, pp. 849–859.

Voon, K.C., Ingham, J.M., Davidson, B.J., “In-Plane Seismic Response of Reinforced Concrete Masonry”, Proc., 12th Int. Brick/Block Masonry Conference, Madrid, 2000, pp. 921-935.

Voon, K.C., Ingham, J.M., “Shear Strength of Concrete Masonry Walls”, Proceedings, 7th Australasian Masonry Conference, Newcastle, 2004, pp.102-111.

Voon, K.C., Ingham, J.M., “Experimental Study of Partially Grouted Concrete Masonry Walls With Openings”, Proc., 10th Canadian Mas. Symposium, Banff, 2005, pp. 722-731, also on CD.

Zhuge, Y., Mills, J., “On Behaviour of Partially Reinforced Masonry Under Simulated Earthquake Load”, Proc., 12th Int. Brick/Block Masonry Conf., Madrid, 2000, pp.1985-1992.