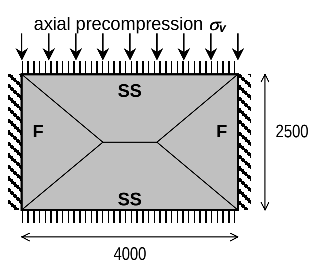
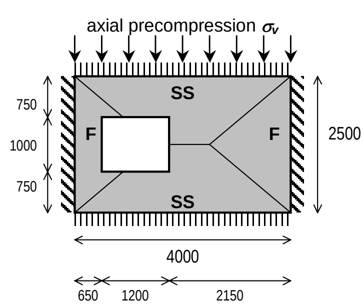
1. BACKGROUND

In order to develop "displacement-based" design rules as an alternative to the traditional "force-based" design guidelines currently used for the seismic design of unreinforced masonry buildings in Australia, a 3-year research project is being undertaken jointly between the Universities of Adelaide and Melbourne. A key aspect of this project is the out-of-plane response of walls in two-way bending. This paper presents the preliminary test results of quasi-static cyclic tests conducted on eight full-scale clay brick masonry walls. The results of these tests are characterised in terms of their overall load versus displacement response, static versus cyclic strength, inelastic displacement capacity and damping. The implications of these tests for seismic loading of unreinforced masonry buildings are also considered.

2. CYCLIC TEST PROGRAMME AND RESULTS

The generic test configuration, wall geometry and boundary conditions for the 8 test walls are shown in Figure 1. Note that three different values of precompression were used over the testing programme and that the vertical edges of all walls were built in to the short return walls to provide a high degree of rotational restraint along the vertical edges. The top and bottom edges essentially behaved as "pinned" connections except in wall six where the top edge was laterally unsupported. The average material properties for each of the test walls are given in Table 1 where it can be seen that the quality of masonry was reasonable for a mortar mix of 1:2:9 (cement:lime:sand).

Wall	*Wall Geometry and Support Conditions*	*Precompression (σ_v)*
1		0.10 MPa
2		0 MPa
3		0.10 MPa
4		0.05 MPa
5		0 MPa

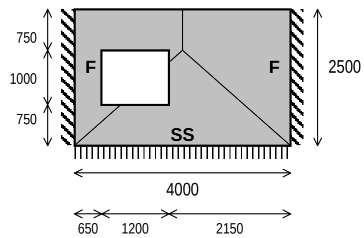
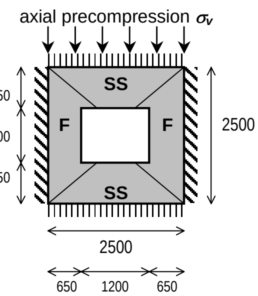
6		N/A Top edge is unsupported.
7		0.10 MPa
8		0 MPa

Figure 1. Test Set-up and Wall Geometry

Table 1 – Material Properties

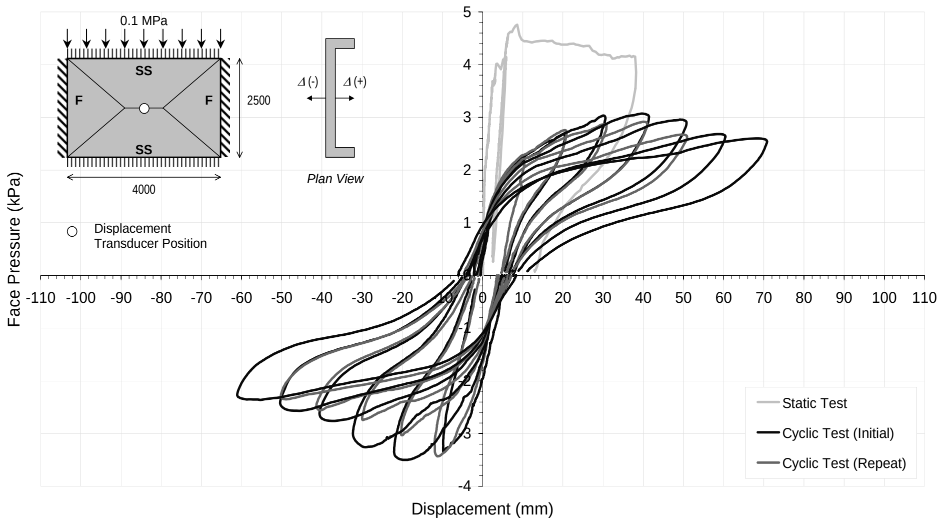
	Mean	COV
Brick unit:		
Flexural tensile strength, f_{ut}	3.55 MPa	0.27
Young's modulus	52,700 MPa	0.35
Masonry:		
Flexural tensile strength, f_{mt}	0.61 MPa	0.19
Compressive strength, f_{mc}	16.0 MPa	0.14
Young's modulus	3540 MPa	0.41

The walls were loaded using airbags. To generate cyclic loading, the airbags on the "outside" face of the test wall were first inflated until the wall's displacement (in the positive direction in Figure 2) reached the target displacement at which point the airbag pressure was released with the load gradually dropping back to zero. The airbag on the "inside" face of the test wall was then inflated until the negative direction target displacement was reached at which point the airbag pressure was released. This process was typically repeated for two complete cycles for each target displacement, with the target displacement values being progressively increased until the displacements approached the wall thickness of 110 mm. A more complete description of the experimental test program is given in Vaculik and Griffith (2005).

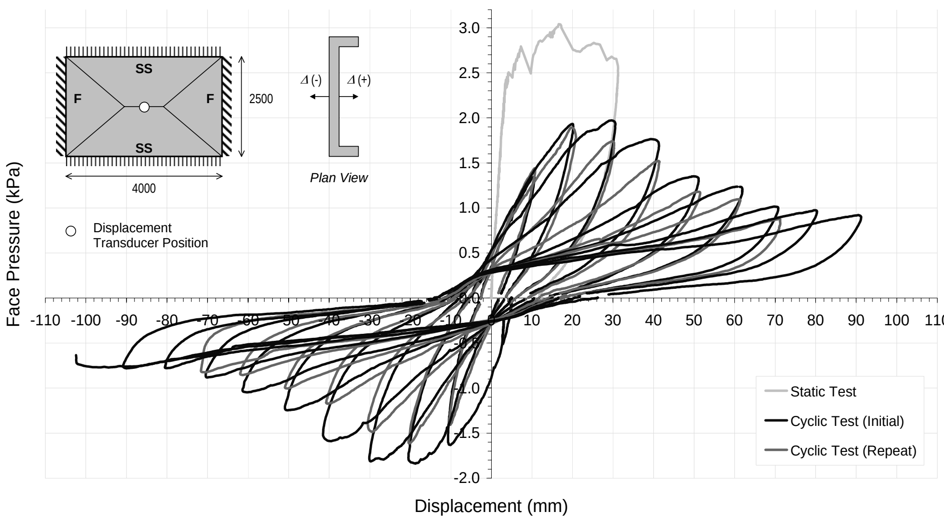
The generic load-deflection characteristics for the walls in this study are now discussed. On the first half-cycle of loading, when the wall was in its "new" (uncracked) condition, the load increased rapidly to its ultimate static strength, F_u, at a corresponding displacement, Δ_u, of between 5 mm and 15 mm after which the load tended to reduce; in some cases quite rapidly. All subsequent loading cycles exhibited smaller maximum strengths although a residual (post-cracking) maximum strength, F_r, was maintained for quite large displacements, in some cases as much as 100 mm! If one can accept the loss of static strength, these walls exhibit quite large displacement capacity and a relatively "ductile" load-deflection response. For example, the hysteresis loops for wall 2 at ± 90 mm of displacement are essentially characteristic of a bilinear inelastic system with a corresponding equivalent viscous damping ratio of 12.7%. The damping ratios for all eight walls were calculated and found to be in the range of 12.7% - 17.6% (refer Table 2) from which it can be seen that the walls exhibit good energy dissipating ability if they are allowed to displace beyond their static (elastic) strength limit state. Note, the damping ratio values were observed to be relative constant for each wall over the entire range of displacements that they were tested over and are substantially greater than the 5% damping value assumed for seismic design.

Another interesting aspect of wall response was the ratio of the post-cracked residual wall strength, F_r, at displacements in excess of 70 mm (after many cycles of loading) divided by the ultimate static wall strength, F_u. This ratio is given in column 3 of Table 2 for each wall and it can be seen that the walls all had a post-cracking strength which was in the range of 21% - 57 % of their ultimate static strength with the drop-off being greatest for the walls with zero precompression. It should also be noted that as the wall displacements became large the hysteretic behaviour became less symmetric. This was due to progressive damage at the return wall connections at large displacements which meant that when the load was pushing outward (in the negative displacement direction) then the connection between the return walls and the test wall tended to be in tension so that progressive loss of integrity meant a loss of stiffness and strength. In contrast, loads pushing inwards (in the positive displacement direction) created a "compression" reaction at the return wall support so that progressive damage to the connection had a much less pronounced effect on the wall strength. The hysteresis loops for wall 7 were especially asymmetric. This was because a "line failure" crack developed at one of the return wall connections which allowed the test wall to deflect in primarily vertical bending at that end when pushed in the negative (outwards) direction. Hence, the maximum negative wall displacement actually occurred near the return wall position, not where the displacement transducer was located near the middle of wall 7.

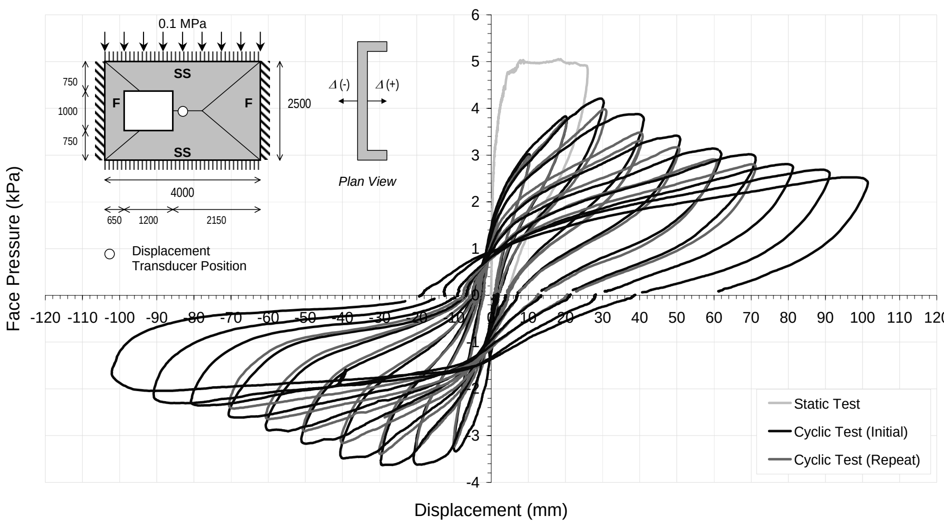
It is difficult to compute displacement ductility values for this type of system using conventional methods but if one divides the wall displacement Δ_r where the residual post-cracking strength is reliably sustained for multiple cycles (e.g., Δ_r = 90 mm for wall 2) by Δ_u, the displacement where the static ultimate strength was attained (about 15 mm for wall 2), we get an indication of the inelastic displacement capacity ratio for each wall. These calculations suggest that wall 2 possesses an inelastic displacement capacity of approximately 6 times Δ_u, which is counter to conventional wisdom that says unreinforced masonry buildings and their components possess little ductility. Inelastic displacement capacity values using this approach have been calculated for all eight walls and as shown in column 4 of Table 2 range from 5 to 12. Note, the choice of what residual force level is suitable for this calculation is somewhat arbitrary and if we instead used the displacements at which point the hysteretic strength has reduced to 75% of F_u as is commonly done for concrete design then the ratios would be smaller but still significantly larger than 1. Hence, these tests indicate that the walls have a substantial displacement capacity.



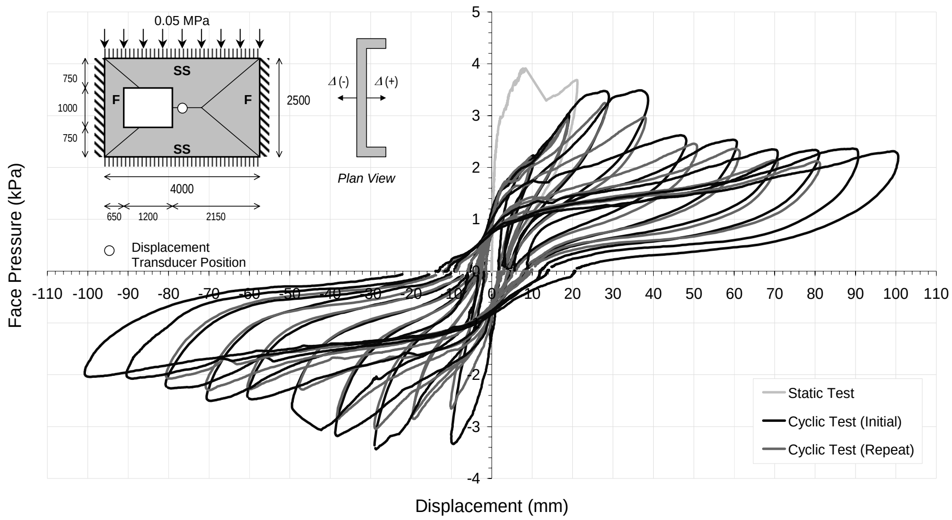
(a) Wall 1



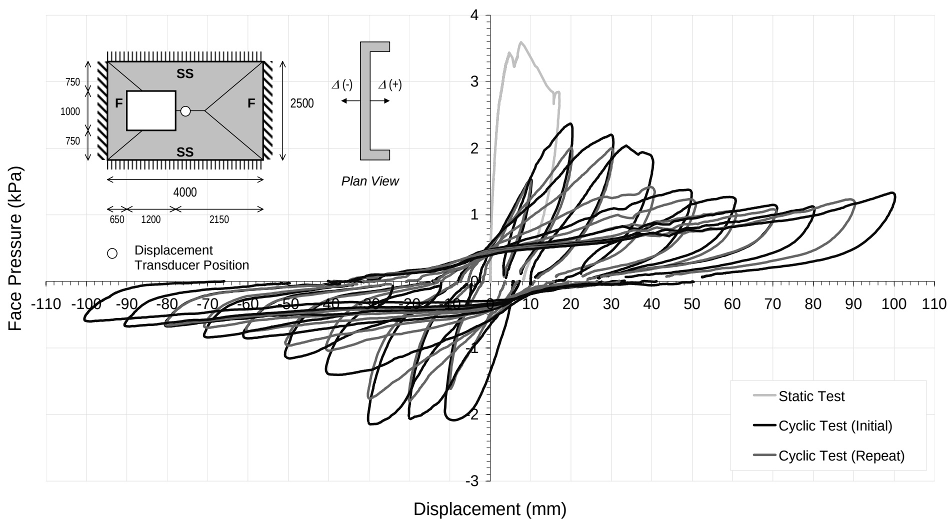
(b) Wall 2



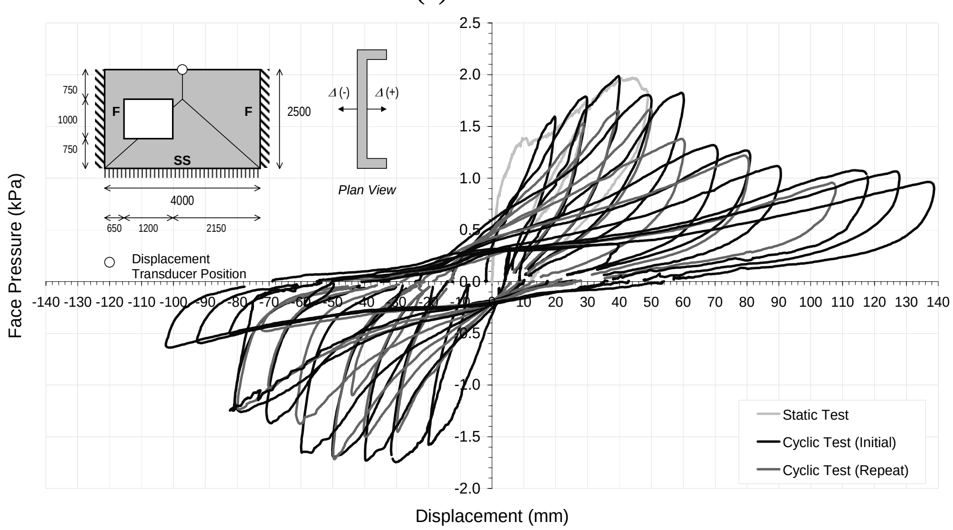
(c) Wall 3



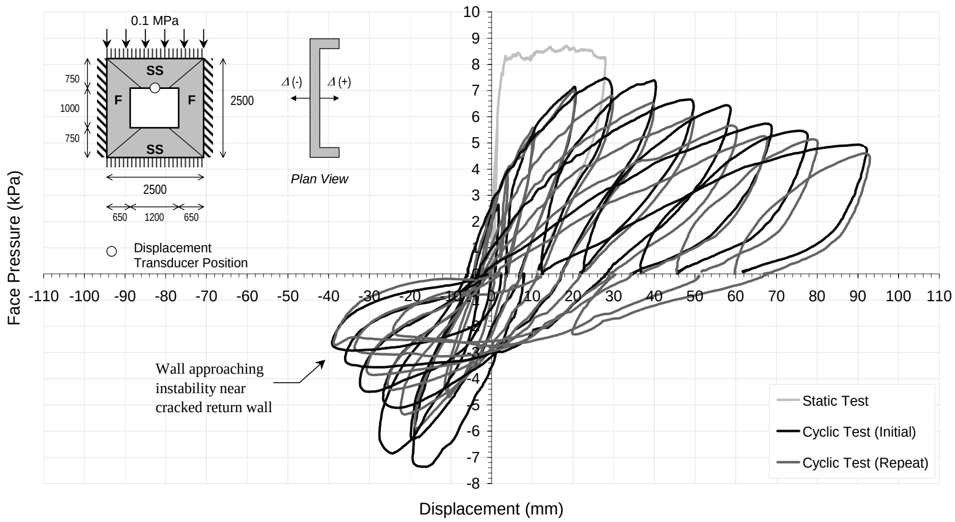
(d) Wall 4



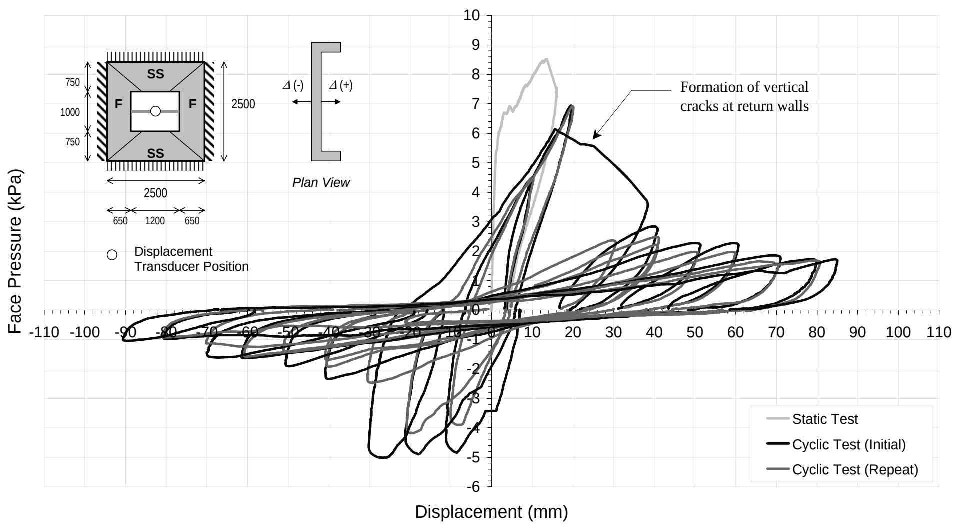
(e) Wall 5



(f) Wall 6



(g) Wall 7



(h) Wall 8

Figure 2. Load versus deflection results for full-scale masonry walls.

Table 2. Wall test results and calculations.

Wall	Equivalent viscous damping ratio	Strength loss ratio F_r/F_u	Inelastic displacement capacity Δ_r/Δ_u	capacity/demand force ratio $F_{capacity}/F_{demand}$
1	13.1 %	2.6/4.70 = 0.56	70/10 = 7	3.20/2.0 = 1.6
2	12.7 %	1.0/3.04 = 0.33	90/15 = 6	2.85/2.0 = 1.42
3	14.2 %	2.8/5.04 = 0.56	100/8 = 12	2.09/2.0=1.04
4	12.3 %	2.2/3.91 = 0.56	100/8 = 12	1.97/2.0 = 0.98
5	16.7 %	1.2/3.59 = 0.33	100/8 = 12	1.85/2.0 = 0.92
6	13.7 %	1.0/1.98 = 0.51	120/14 = 8	0.9/2.9 = 0.45
7*	15.3 %	5.0/8.70 = 0.57	90/12 = 7	4.79/2.0 = 2.40
8	17.6 %	1.8/8.5 = 0.21	80/16 = 5	3.91/2.0 = 1.96

*Wall 7 values computed based on positive direction loading only.

3. DISCUSSION AND CLOSING REMARKS

When trying to assess the implications of these results for current design practice in Australia, it is useful to compare the seismic force capacity and demand for each wall in accordance with the current design guidelines. The wall force capacity, $F_{capacity}$, for each wall in Table 2 was calculated using the guidelines in the Australian Masonry Structures code, AS 3700 (SA 2001) and assuming the default material property values in the code and edge restraint factors of 0.5 for R_{f1} and R_{f2}. The seismic force demands, F_{demand}, were calculated using the earthquake design category 1 provisions in the ballot version of AS 1170.4 (SA 2005) resulting in a seismic load demand of 2.0 kPa for each wall – using self weight W_i = 20 kN/m^2 and $F_i = 0.1\ W_i$. The ratio of the seismic force demand divided by capacity is given in column 5 of Table 2 for all eight walls. Based on these values, half of the eight walls have sufficient strength capacity to resist the seismic design loads, two walls (3 and 4) have capacity approximately equal to demand and two walls (5 and 6) have force capacities that are insufficient to meet the seismic force demands.

Based on the preliminary results of quasi-static cyclic tests of eight full-scale unreinforced brick masonry walls, it appears that these walls have a substantial inelastic displacement capacity that is being ignored under the current force-based design guidelines. As shown by sample calculations in this paper, the seismic force demands on masonry walls can be greater than their design force capacity. The implications of this are that in some instances during a “design magnitude” earthquake, the walls are likely to exceed their static force capacity. The question remains, however, will they become unstable and become a threat to life safety. We believe that these walls have a substantial inelastic displacement capacity beyond the displacements at which their ultimate strengths are reached. However, we must be able to estimate the inelastic displacement demand in order to assess compliance, or otherwise, of these walls with regard to the “life safety” performance criteria. Further work along these lines is continuing with shaking table tests planned for 2006 to ascertain how representative the quasi-static cyclic tests are of actual dynamic response in order to develop suitable design techniques for estimating the inelastic flexural displacement demand on masonry walls imposed by earthquake induced ground motion.

4. REFERENCES

Standards Australia. “AS 3700 - Masonry Structures, Homebush, NSW, 2001.

Standards Australia. “Structural design actions, Part 4: Earthquake actions in Australia (Revision of AS 1170.4 – 1993)”. Ballot Version of Draft Australian Standard, PDR-5212, Homebush, NSW, 2005.

J Vaculik and MC Griffith, “Flexural strength of unreinforced clay brick masonry walls,” *Proceedings of 10th Canadian Masonry Symposium*, Banff, Alberta Canada, (2005).