FLEXURAL STRENGTH OF UNREINFORCED CLAY BRICK MASONRY WALLS

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ABSTRACT

This paper presents the results of an experimental test program on eight full-scale unreinforced brick masonry walls. Three different levels of vertical precompression were used in the tests and all walls were 2.5 m tall with six being 4 m long and two being 2.5 m long. Six of the eight walls also each had a single window opening (nominally 1.2 m × 1.0 m).

The walls were laterally supported along all four edges. The top and bottom edges were "simply-supported" while the vertical edges were supported by short return walls, which were restrained from rotation so that the two vertical edges could reasonably be considered to be "fixed" rotationally. The walls were loaded using a system of airbags placed between the wall and a reaction frame. The forces going into the reaction frame were recorded using load cells and these loads were plotted versus the mid-wall displacement to characterise the flexural behaviour of each wall. From these plots the ultimate strength of each wall was determined and compared to predictions using the virtual work method in combination with the AS 3700 moment expressions and recently published expressions by Willis [14, 15] for horizontal and diagonal bending moment capacities. The paper concludes with a summary of the accuracy of the analytical predictions and implications for seismic design of brick masonry walls.

KEYWORDS: unreinforced brick masonry, walls, bending, experimental, tests

INTRODUCTION

Research into the seismic risk posed by URM buildings in North America, Europe and New Zealand [1, 3, 4, 9] has highlighted the need for improvements in understanding of how URM buildings behave under earthquake loading and corresponding improvements in the earthquake design procedures for URM construction.

Significant improvements in the design methods available to calculate the static strength of brick masonry walls supported on all four sides and subject to out-of-plane loads have been achieved [8]. The improvements, based on a virtual work, "force-based" approach, still assume failure occurs once a wall has reached its ultimate strength, and this occurs in bending at very small displacements, of the order of 2 to 5 mm. This restriction, whilst appropriate for wind loading, has been shown to be extremely conservative for seismically loaded URM walls [2, 4, 10, 12], which do not collapse until the bending displacements approach the thickness of the wall, typically of the order of 100 mm for single leaf clay brick masonry. Some progress has also been made in modelling the non-linear dynamic response of URM buildings [7, 11]. However, there is a wide variation in the Young's modulus and tensile and compressive strengths of the masonry. Force-based (FB) design, using a low flexural strength (mean – 1.65 × standard deviation) to account for these uncertainties, can be unduly conservative. Hence, work is now underway to extend a newly developed displacement-based procedure for the seismic assessment of URM walls in one-way vertical bending [5] for application to walls in two-way bending. A key step in this development is a better understanding of the full load-deflection behaviour (up to and including collapse) of URM walls in two-way bending.

EXPERIMENTAL TESTS

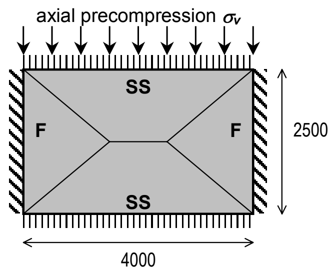
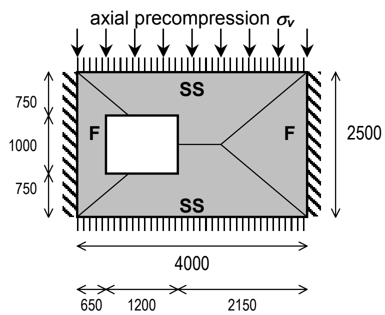
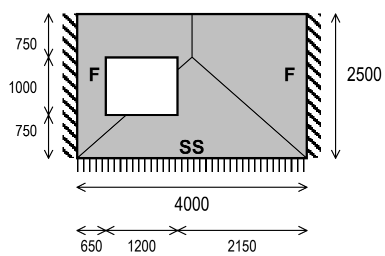
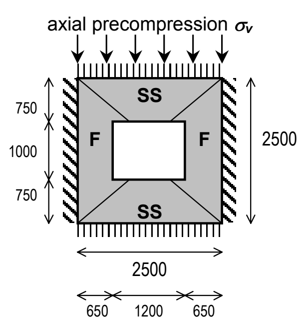
Eight full-scale clay brick masonry walls were subjected to quasi-static loading as part of this project to investigate the load-deflection behaviour of unreinforced masonry (URM) walls well beyond the point of maximum strength. Of course, the static strength of the walls was obtained as a bi-product from these tests and is the main focus of this paper. The geometry of the eight wall test specimens is given in Table 2 where it can be seen that two solid walls and six walls with openings were considered. Walls 1 and 2 were each 4 m long and 2.5 m tall and tested with vertical precompression of 0.10 MPa and without vertical pre-compression, respectively. Walls 3 to 5 were also 4 m long by 2.5 m tall, but each wall also had a window opening located 650 mm away from the left-end of the wall. These walls were subject to vertical precompression of 0.10 MPa, 0.05 MPa and 0 MPa, respectively. Wall 6 was identical to Wall 5 except that its top edge was free. Walls 7 and 8 were each 2.5 m long by 2.5 m tall and had a window opening located symmetrically between each end. A precompression of 0.10 MPa was also applied to Wall 7.

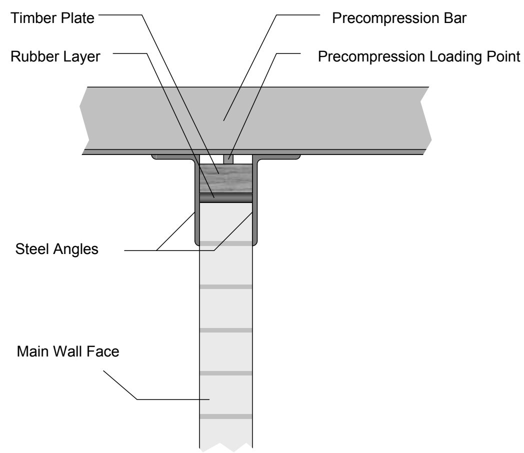
The walls were constructed by qualified bricklayers using 10-hole cored clay brick units with nominal dimensions of 230 mm × 76 mm × 110 mm (length 'l_u' × height 'h_u' × width 't_u'), typical mortar joint thickness t_m of 10 mm, and a 1:2:9 mortar mix (cement:lime:sand) that was bucket batched to minimise mortar batch variability. The engineering properties (mean values and coefficients of variation, COV) for the brick units and masonry are listed in Table 1, where it can be seen that the quality of the masonry is typical for the mortar mix used.

Table 1 – Material Properties

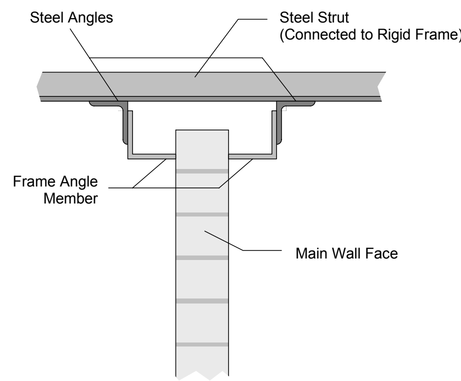
	Mean	**COV**
Brick unit:		
Flexural tensile strength, f_{ut}	3.55 MPa	0.27
Young's modulus	52,700 MPa	0.35
Masonry:		
Flexural tensile strength, f_{mt}	0.61 MPa	0.19
Compressive strength, f_{mc}	16.0 MPa	0.14
Young's modulus	3540 MPa	0.41

Table 2 – Wall Geometry and Boundary Conditions

<table>
<tr><th>Wall</th><th>Wall Geometry and Support Conditions</th><th>Precompression (σ_v)</th></tr>
<tr><td>1</td><td rowspan="2">
</td><td>0.10 MPa</td></tr>
<tr><td>2</td><td>0 MPa</td></tr>
<tr><td>3</td><td rowspan="3">
</td><td>0.10 MPa</td></tr>
<tr><td>4</td><td>0.05 MPa</td></tr>
<tr><td>5</td><td>0 MPa</td></tr>
<tr><td>6</td><td>
</td><td>N/A
Top edge is unsupported.</td></tr>
<tr><td>7</td><td rowspan="2">
</td><td>0.10 MPa</td></tr>
<tr><td>8</td><td>0 MPa</td></tr>
</table>



(a) with pre-compression



(b) without pre-compression

Figure 1 – Support Details at Top of Wall

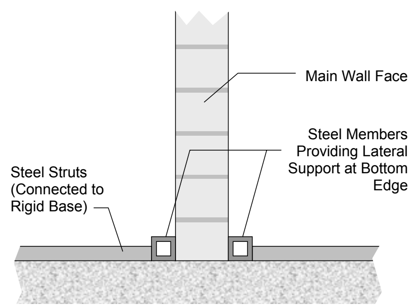
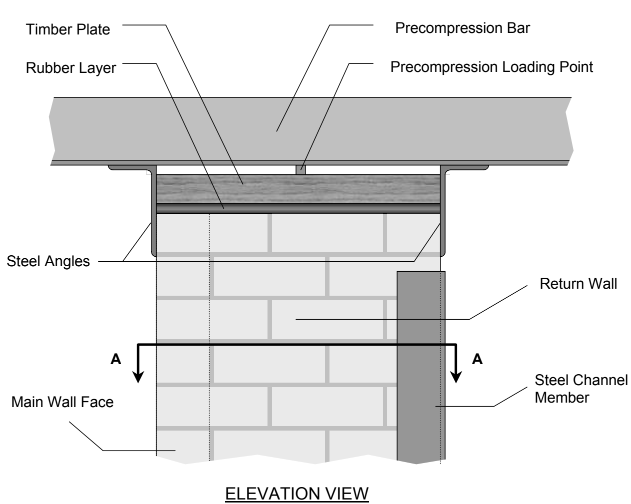
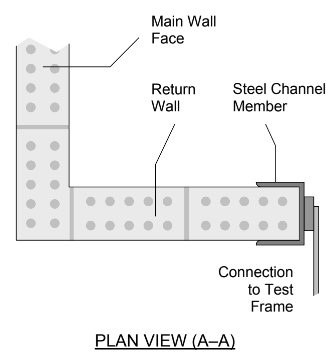


Figure 2 – Support Details Along Bottom Edge of Wall



(a) Side elevation view



(b) Plan view

Figure 3 – Support Details Along Vertical Edge of Return Wall

An important aspect of the experiments was the care taken to simulate simple-supports at the top and bottom edges of the walls and fixed-supports along the vertical edges. Figure 1 illustrates the method used to laterally restrain the top edge of walls, with and without pre-compression. The supports shown in Figure 1 continued along the entire length of the top of each wall. Lateral support was also provided along the base of each wall (refer Figure 2) to ensure that the test specimens did not slip at the base.

In order to provide a full moment connection along the vertical edges of each wall the free end of each return wall was "clamped" by a channel section which was in-turn restrained from moving laterally by bracing back to the test frame (refer Figure 3). This enabled the masonry to develop its full horizontal bending capacity along the intersection of the main and return walls as evidenced by the vertical cracks that developed at this location during the tests. An overview of the test set-up for Wall 1 is shown in Figure 4.

Figure 4 – Overview of Wall Test Set-up

The walls were loaded using a system of airbags placed between the wall and a reaction frame. The forces going into the reaction frame were recorded using load cells and these loads, divided by the wall surface area, are plotted against the wall deflections in Figures 5 to 7. In all walls, the actual cracking patterns were very similar to the idealised cracking patterns shown in Figures 5 to 7. The location on the wall for which the deflection was plotted is indicated by a white circle in each figure with, in most instances, the location being the point of maximum deformation in the wall. The following observations can be made from the load-displacement curves shown.

- In every case, there is very clear evidence of increased strength with increased vertical pre-compression, as shown by comparisons of walls 1 and 2 (Figure 5), walls 3, 4 and 5 (Figure 6) and walls 7 and 8 (Figure 7). This is mainly due to the increased torsional bed joint resistance due to the greater vertical compressive stress on the bed joints, which contributes to the moment capacity of the masonry in horizontal and diagonal bending.
- Significantly, no specimens experienced a sudden drop in load resistance after reaching the ultimate strength, instead providing continued resistance with increased deformation and thus exhibiting ductility and displacement capacity well in excess of the displacement at which the walls cracked (typically between 2 and 5 mm). Good ductility and displacement capacity are highly desirable with respect to earthquake loading.
- The walls with openings (3 and 5) have greater strength than the corresponding walls without openings (walls 1 and 2), as shown by Figures 5 and 6. This observation is a result of the openings near the mid-span of the walls causing a substantial reduction in the external virtual work (load multiplied by virtual displacement) imposed on these walls compared to the corresponding walls without openings.
- Wall 6 was unsupported at the top edge and exhibited a significantly larger displacement at the point of ultimate strength compared with the other walls. This is due to wall 6 having a longer effective span and thus, requiring a greater central displacement before it could develop full rotation capacity at the vertical edges.
- In all cases, the walls unload inelastically, resulting in an incomplete recovery of deformation, which is indicative of significant energy dissipation capacity – another desirable characteristic with regard to earthquake loading. Interestingly, this contrasts with the "elastic" unloading behaviour reported by Doherty et al (2002) [5] for URM walls in one-way vertical bending.
- Short walls 7 and 8 showed an increase in strength compared with long walls 3 and 5, respectively, due to their shorter span.

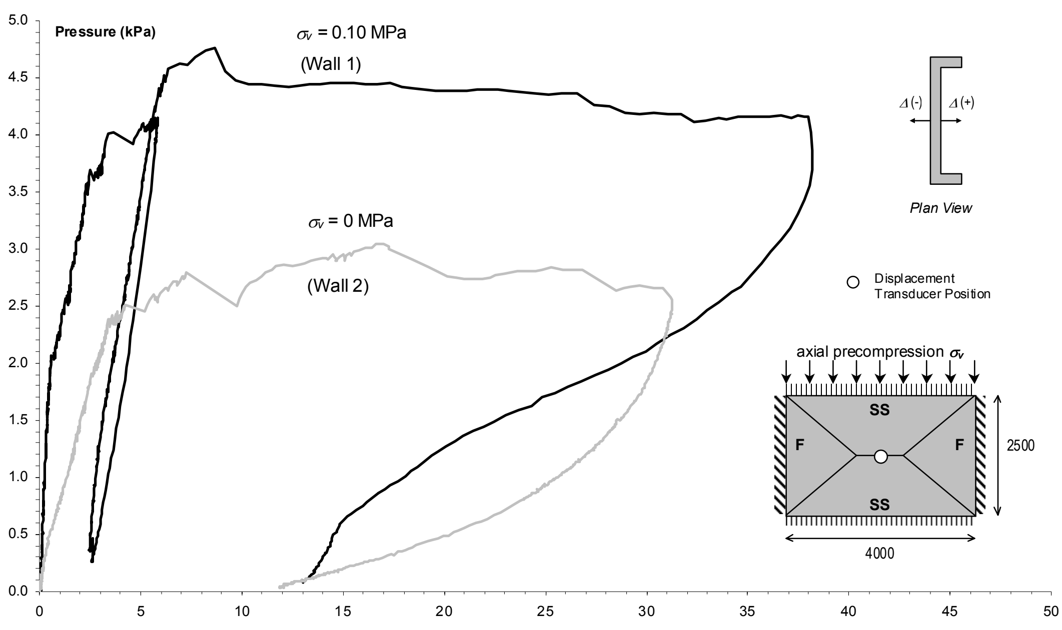


Figure 5 – Long Walls Without Openings

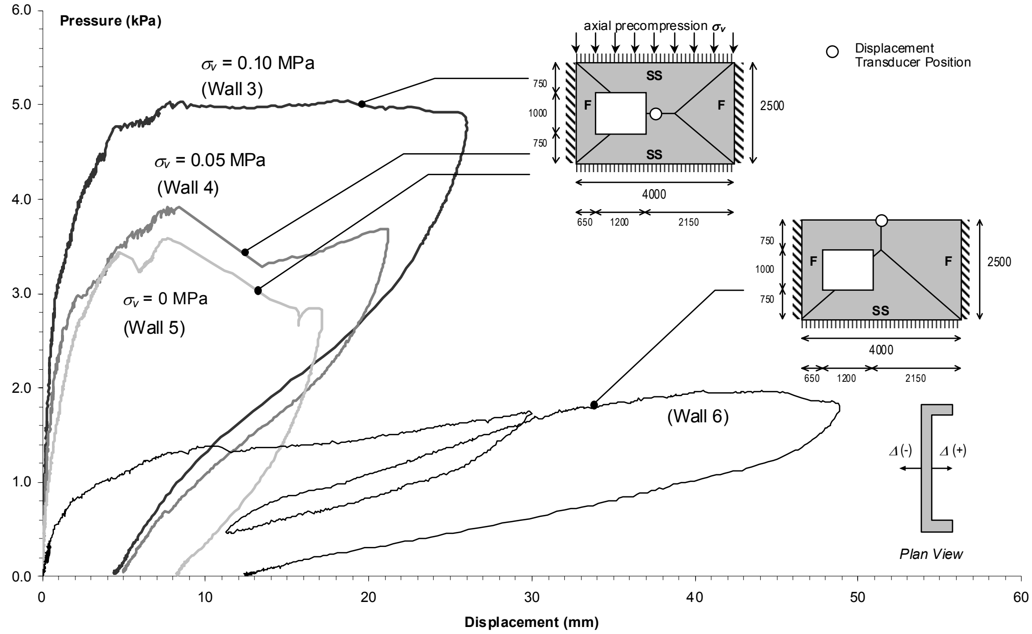


Figure 6 – Long Walls With Window Openings

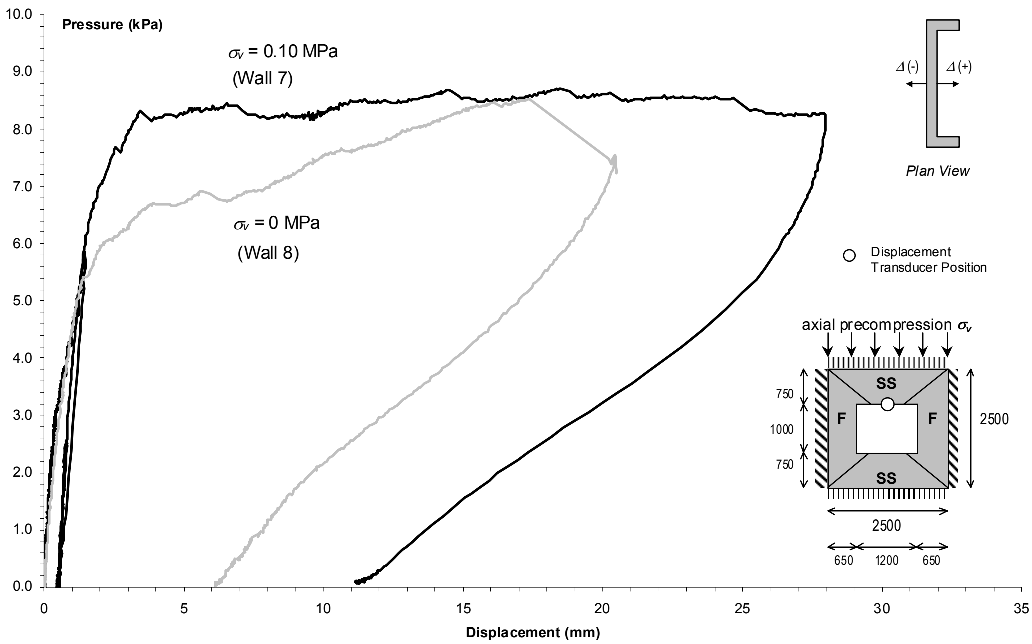


Figure 7 – Short Walls With Window Openings

ANALYTICAL PREDICTIONS

In this section, the experimental results for the eight walls are compared to the predictions of strength given by the current design expressions given in the Australian Masonry Structures

Code, AS3700 [16], and recently published expressions by Willis [14, 15]. The expression used to predict the bending strength of URM walls in AS 3700 is

$$w_c = \frac{2a_f}{L_d^2} \cdot \left(k_1 \cdot M_h + k_2 \cdot M_d\right) \qquad \text{Equation 1}$$

where a_f, k_1 and k_2 are coefficients based on the wall geometry and boundary conditions, L_d is the design length of the wall, and M_h and M_d are the horizontal and diagonal bending moment capacity of the masonry wall, per unit length of crack line. Equation 1 is based on the virtual work method of analysis, whereby at the point of maximum strength the external work done on the wall by the loads is equal to the internal work done by the internal bending moments acting along all the vertical and diagonal crack lines in the wall.

The AS 3700 expressions for the respective moment capacities are

$$M_h = \text{lesser of} \begin{cases} \phi \cdot k_p \cdot \left(0.44 \cdot f_{ut} + 0.56 \cdot f_{mt}\right) \cdot Z_d & \textit{line failure} \quad \text{Equation 2a} \\ 2.0 \cdot \phi \cdot k_p \sqrt{f_{mt}} \cdot \left(1 + \dfrac{f_d}{f_{mt}}\right) \cdot Z_d \le 4.0 \cdot \phi \cdot k_p \sqrt{f_{mt}} \cdot Z_d & \textit{stepped failure} \quad \text{Equation 2b} \end{cases}$$

$$M_d = \phi \cdot f_t \cdot Z_t \qquad \text{Equation 3}$$

where $f_t = 2.25\sqrt{f_{mt}}$ is the equivalent flexural strength of the masonry along a diagonal crack line, expressed in terms of the masonry tensile bond strength f_{mt} and the effective section modulus of the masonry along a diagonal crack Z_t. (Note: f_d is the vertical compressive stress in the wall at its mid-height which includes the vertical precompression, σ_v, applied at the top of a wall.) In subsequent calculations mean values were used for the material properties and values of 1 were assigned to the capacity reduction factor, ϕ, and perpend factor k_p.

In recent publications [15] alternative expressions were proposed for the moment capacity of URM in horizontal and diagonal bending. These are reproduced as Equations 4 and 5 respectively, in a moment per unit length of crack formulation.

$$M_h = \text{lesser of} \begin{cases} \dfrac{1}{2\left(h_u + t_m\right)}\left[\left(f_{ut} - \nu \cdot f_d\right) \cdot h_u \dfrac{t_u^{\,2}}{6}\right] & \textit{line failure} \quad \text{Equation 4a} \\ \dfrac{1}{h_u + t_m}\left[\tau_u k_b \cdot 0.5\left(l_u + t_m\right) \cdot t_u^2\right] & \textit{stepped failure} \quad \text{Equation 4b} \end{cases}$$

$$M_d = \frac{\sin\varphi}{h_u + t_m}\left[\left(\sin\varphi\right)^3 \tau_u k_b 0.5\left(l_u + t_m\right)t_u^{\,2} + \left(\cos\varphi\right)^3\left(f_{mt} + f_d\right)\frac{0.5\left(l_u + t_m\right)t_u^{\,2}}{6}\right] \qquad \text{Equation 5}$$

where τ_u is the ultimate shear bond stress of a bed joint given as $\tau_u = 1.6 f_{mt} + 0.9 f_d$, φ is the slope of a diagonal crack line which can be determined from unit geometry, k_b is a numerical factor used to calculate the shear stress due to torque on a rectangular cross section [13] and is equal to 0.214 for the masonry units used in these walls.

The AS 3700 and Willis predictions are compared with experiment in Figure 8, where the predictions made using Equations 4 and 5 lie closer to the line of equality than the code predictions. Table 3 lists the results of the calculations and shows that, on average, the AS 3700 expressions over-predict the experimental strength by 25% while the Willis expressions over-predict the experimental results by only 2%. The coefficients of variation for w_{pred}/w_{exp} were essentially the same for the code and Willis expressions, i.e. 26% and 25%, respectively. That the AS 3700 expressions over-predict the bending strength is somewhat surprising since comparisons to test results from other researchers (over 60 wall tests) has indicated that it generally underestimates bending strength. The reasons for this are still being investigated.

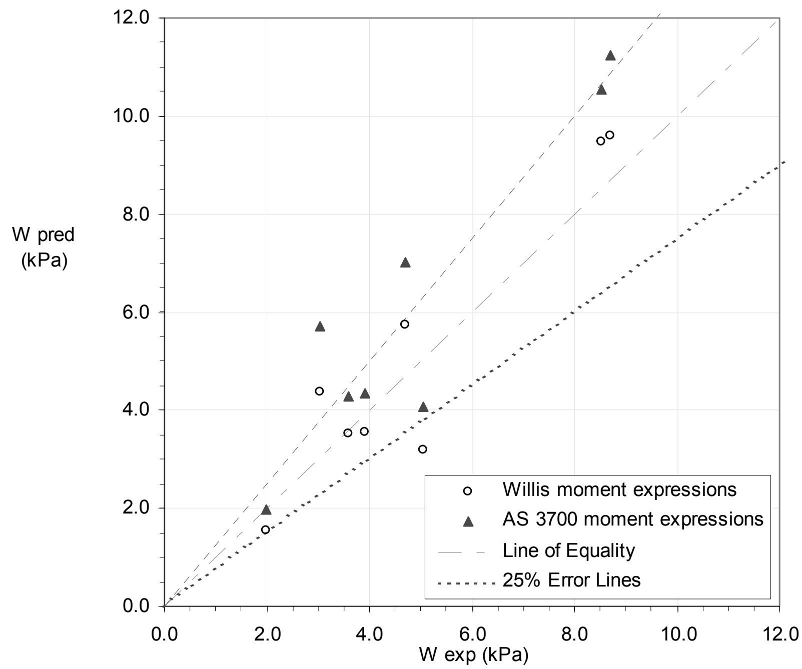


Figure 8 – Comparison of Analytical and Experimental Strength Predictions

Table 3 – Analytical Results of the AS 3700 Virtual Work Approach

Wall	Experimental Test Result	AS 3700 Moment Expressions		Willis Moment Expressions	
	w_{exp} (kPa)	w_{pred} (kPa)	w_{pred} / w_{exp}	w_{pred} (kPa)	w_{pred} / w_{exp}
1	4.70	7.02	1.49	5.74	1.22
2	3.04	5.72	1.88	4.39	1.44
3	5.04	4.07	0.81	3.20	0.63
4	3.91	4.34	1.11	3.55	0.91
5	3.59	4.29	1.20	3.52	0.98
6	1.98	1.97	0.99	1.55	0.78
7	8.70	11.24	1.29	9.60	1.10
8	8.52	10.54	1.24	9.47	1.11
		Mean Values:	1.25		1.02

SUMMARY AND CLOSING REMARKS

Static airbag tests conducted on eight unreinforced brick masonry walls were described in this paper. Of the eight walls, six contained window openings. The experimental data indicates that face-loaded masonry walls have some ductility and a substantial displacement capacity beyond the cracking displacement. This suggests that a displacement-based methodology may be useful for assessing the seismic capacity of URM walls in 2-way bending. Finally, new expressions for diagonal and horizontal moment capacity in URM walls have been shown to give reasonably accurate predictions of wall strength when used in the AS 3700 virtual work methodology. However, these expressions require further investigation through comparisons with the experimental results of other researchers.

ACKNOWLEDGEMENTS

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