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Two-way cavity clay brick walls tested in-situ

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Abstract

This research is aimed at producing experimental data in relation to the out-of-plane behavior of two-way spanning unreinforced clay brick cavity walls, which is currently a significant area of shortcoming in seismic assessment of masonry buildings. Destructive out-of-plane airbag tests were conducted on a total of 10 two-way spanning walls (6 cavity, 4 solid) and 1 vertically spanning cavity wall in 3 low-rise properties located in Darlington, South Australia. The load was applied on the face of one leaf and the lateral displacements of both wall leaves were measured. Companion material testing was done both in-situ and in laboratory to aid in the calculation of theoretical wall strength. It was observed that the as-built cavity wall ties possessed significant compressive strength that maintained the cavity by promoting two-way out-of-plane mechanism in the unloaded wall leaf, i.e. the wall ties strength often exceeded single-leaf two-way spanning wall strength. Calculations showed that the strength of cavity walls as a whole exceeded the theoretical calculations made based on an idealized method available in seismic codes. The underestimation of the wall strength by codes and analytical studies were attributed to factors that included ignoring wall rendering and potentially conservatisms associated with the formulation of the methods.

Keywords: cavity wall; unreinforced masonry; two-way bending; airbag; in-situ

1. Introduction

A major problem faced by the practicing structural engineers is the seismic evaluation of out-of-plane loaded URM cavity walls. A significant proportion of the previous related research has focused on solid walls, mostly one-way vertically spanning [1,2] and to a lesser extents two-way spanning walls [3,4]. Recent research has been reported on cavity wall response in one-way bending condition [5], but no documented laboratory or in-situ testing of cavity wall in two-way bending exists in the masonry literature.

Cavity construction includes walls built in two leaves, with a gap of typically 25-75 mm between the leaves. Usually a number of slender bent steel wall ties bridge the gap. Although the newer construction includes ties designed to maximize bonding capability, the wall ties in older construction are simply U-shaped thin, e.g. 2-4 mm dia, bent plain steel bars. It is known that the tensile force carrying capacity of the ties is limited to friction due to no adhesives being used in installation. Furthermore, the ties slenderness limits their compressive capacity, although no research has been done to properly study the effectiveness of ties in maintaining the gap. Consequently, wall leaves are typically evaluated individually and irrespective of the wall ties resulting in a potentially overly conservative seismic assessment.

The lack of understanding of the capacity of the wall ties creates further problem when designing a seismic retrofit for inadequately built walls. The limited access to cavity prevents the application of direction-sensitive surface treatments, e.g. near-surface-mounted (NSM) fibre-reinforced-polymer (FRP) strips, to both faces of individual leaves, and a retrofit design needs to be carried out considering applying FRP on external faces only (one face of each leaf). A knowledge of the strength of the ties will help establishing if an integral wall response under cyclic loading can be achieved.

The research reported herein was aimed at producing experimental data obtained through in-situ testing on cavity URM walls. A summary of the tests undertaken on a total of 10 two-way spanning walls (6 cavity, 4 solid) and 1 vertically spanning cavity wall in 3 different properties located in Darlington, South Australia are presented. The data will be used to evaluate a codified procedure and an existing lab-based model, hence increasing confidence in using the methods by practicing engineers. The data also presents evidence of the adequacy of the wall ties to resist the compressive forces needed to maintain the cavity gap.

2. Case study buildings

Two of the buildings were single-storey suburban houses (Figure 1a and b) estimated to have been built in early 1960s and closely resembling each other in general aspects, e.g. wall thickness, finishes, cavity wall construction details, wall ties, and roof type. The roof of these buildings was mildly pitched, made of timber rafters and sheathing, and overlaid with masonry tiles. Its weight was supported by the outer leaf of the external walls resulting in the inner leaf to be without applied pre-compression. Some of the weight of the roof was carried by internal walls via struts connected to wall top. The structural foundation consisted of concrete strip footings with a wooden flooring. The 3rd building was an early 1980s two-storey house with a URM ground storey and a fibrous cement/steel frame on the top-storey (Figure 1c). The floor was 120 mm concrete slab that distributed the gravity loads over both masonry leaves. The roof had a light-weight metal construction.

(a) Building A (b) Building B (c) Building C

Figure 1: Case study buildings

3. Masonry units, pattern, and cavity details

3.1 Masonry units

Cored brownish clay bricks (2 holes) with dimensions of 230 (L) x 110 (D) x 76 (H) mm (categorized as solid brick by the Australian Masonry Standards, AS3700 [6]) were used in the external leaf of Building A (Figure 2a), but solid yellow frogged clay bricks (Figure 2b) with dimensions averaging 240 (L) x 110 (D) x 76 (H) mm were used on the internal wall leaf.

For Building B, both the internal and external wall leaves were constructed using solid yellow frogged clay bricks (Figure 2c), 230 (L) x 110 (D) x 76 (H) mm in dimension. In Building C, 3-hole brown cored clay bricks (Figure 2e) of the dimensions of 230 (L) x 110 (D) x 76 (H) mm were used.

(a) Outer leaf of Wall A-1

(b) Inner leaf of Wall A-1

(c) Outer leaf of Wall B-2

(d) Inner leaf of Wall B-2

(e) Outer leaf of Wall C1

(f) Outer leaf of Wall C1

Figure 2: Brick types, failure pattern in bond wrench tests, and typical cavity tie

3.2 Masonry pattern and wall rendering

The inner leaf of the external cavity walls and all the internal walls in Buildings A and B had been built with bricks laid on their depth, producing 76 mm leaf thickness, as opposed to the outer leaf and all walls in Building C that had been constructed with bricks laid on their width resulting in 110 mm wall thickness (Figure 3).

The inner face of the internal leaf of cavity walls (i.e. A-1, A-2, B-2, and B-4; typical out-to-out thickness of 110+55(gap)+76+10=250 mm; see Figure 3a) and both faces of the internal single-leaf walls (i.e. A-3, B-3, and B-5; typical thickness of 76+20 mm; see Figure 3b) had a 10-mm low-strength cement plaster finish. The walls in Building C had the original brick surface without rendering.

(a) Wall B-2 (typical external wall in Buildings A and B)

(b) Wall A-3 (typical internal wall in Buildings A and B)

Figure 3: Details of wall construction (Buildings A and B)

3.3 Cavity structure

The wall ties were found to be 4 mm in diameter for Buildings A and B and 3 mm for Building C, with no evidence of corrosion despite proximity (5 km distance) to coast. The ties were u-shaped and spaced irregularly, ranging from 400 mm to 800 mm vertically and horizontally. Visual observations revealed that the cavity were clear apart from the ties except for two occasions that clogs of mortar had filled cavity areas of about half brick surface. Evidence was found of ties that had been bent (see Figure 2f) to fit mortar joints that were at different levels due to the bricks in the two leaves being laid on different orientations.

4. Material properties

Undamaged brick and mortar samples were collected from or near to the test walls and tested in accordance with the Australian Standards to obtain flexural bond strength of the masonry, f_{mt}, among other material properties. The full range of material tests can be found in the related departmental report [7]. The masonry bond strength were found to be on average 0.13 MPa with a CoV of 0.56 from the 44 specimens that were tested with wall rendering removed if existed. The bond strength was found to be on average 0.56 MPa with a C.o.V of 0.25 from the 6 samples that had plaster on their tension side indicating the significance of plaster contribution to cracking strength. The bond strength data for individual walls were used in the analysis reported herein.

5. Test walls

Geometrical properties and boundary conditions of the tested walls have been summarized in Table 1. The wall boundary condition, applied overburden, and direction of loading has been further detailed in the following sub-sections.

Table 1: Test walls

Wall	Length L (mm)	Height h (mm)	Thickness(1) t (mm)		L/h	h/t		Supported Edges(2)		Applied Overburden (kPa)	
			L(3)	O(4)		L	O	L	O	L	O
A-1	2430	2750	76	110	0.9	32.0	22.1	3	4	0	9.5
A-2	4075	2750	76	110	1.5	53.6	37.0	3	4	0	9.5
A-3	3020	2750	76		1.1	39.7		3		4.1	N/A
B-1	4050	2530	110		1.6	36.8		3		6.8	N/A
B-2	3795	2720	110	76	1.4	34.5	49.9	4	2	13.6	0
B-3	3260	2720	76		1.2	42.9		3		4.1	N/A
B-4	3030	2720	76	110	1.1	39.9	27.5	3	4	0	13.6
B-5	3040	2720	76		1.1	40.0		4		4.1	N/A
C-1	3980	2415	110	110	1.6	36.2	36.2	3	4	38	38
C-2	1990	2520	110	110	0.8	18.1	18.1	3	3	20	20
C-3	1790	2400	110	110	0.8	21.8	21.8	2	2	38	38

(1) excluding plaster (2) see also description and relevant figures for support details

5.1 Openings

The walls generally had no openings except for small penetrations for ventilation in some walls, an electricity box in one wall, and a small window in another wall as described herein. Wall A-1 had 4 small (250 x 180 mm) ventilation cavities close to the corners of the walls similar to those visible in Figure 1a. Wall A-2 had similar ventilation openings in addition to a 1.1 x 0.4 m cut-out in the external leaf that had previously housed the mains electricity breaker box. Wall B1 included a single window opening measuring 970 x 610 mm that was framed with a 90 x 45mm pine frame with 19 mm thick plywood panel before testing.

5.2 Top restraint and applied pre-compression

In both Buildings A and B, the top of the inner leaf of the external walls (i.e. Walls A-1, A-2, B-2, and B-4) was loosely connected to the ceiling via a cornice, with the connection providing little or no restrain and these wall leaves carrying no axial load. The top of the exterior leaf was supported by roof rafters (*Figure 4*a), with the rafters seating on a top timber bearing plate that was nailed into the top course of the masonry (*Figure 4*b). Hence, the external wall leaf was load-bearing.

The internal walls of these two buildings, i.e. Walls A-3, B-3, and B-5, were pinned by roof struts and carried a portion of the roof load. Wall B-1 was an external wall of a garage with corrugated metal roof that applied pre-compression directly to the wall. This was pinned to the parallel wall in the other side of the garage via several steel rods.

Both leaves of all walls tested in Building C was supported by the concrete slab, hence the walls being capable of developing vertical arching action. Both wall leaves carried slab weights represented by the overburden ratios in Table 1.

(a) Roof rafters sitting on external wall leave

(b) Nailed connection of the timber plate to external wall leaf

Figure 4: Details of roof rafters sitting on top of the exterior walls (Building A)

5.3 Airbag loading and boundary conditions along the vertical edges

The direction of airbag loading for all walls except Wall B-2 was opposite to the direction of the cross walls (*Figure 5*). Walls A-1 and A-2 were adjacent and collectively formed the entire southern face of Building A. A cross wall separating these two walls were connected only to the internal wall leaf, hence the external leaf of these two walls were continuous as depicted in *Figure 5*a. Wall A-3 (*Figure 5*b) was connected to cross walls at both ends.

Wall B-1 spanned between a masonry corner and a door as depicted in *Figure 5*c, with significant pre-cracking existing in the spandrel above the door suggesting that the vertical wall edge at this location had little or no restraint. Single unit width masonry piers were built hard against the wall as depicted in *Figure 5*c, although no positive connection existed between the wall and piers. The boundary conditions along vertical edges of Walls B-2 and B-4 for both the inner and outer wall leaves (*Figure 5*d and 5f) were the same as that described for Walls A-1 and A-2 except that or wall B-2 a return wall connected to the inner leaf had a full-height door opening. Walls B-3 and B-5 (*Figure 5*e and g) had the same boundary condition similar as Wall A-3.

Wall C-1 spanned between a single leaf return wall and a glass sliding door as shown in *Figure 5*h. However, removal of the timber cladding of the return wall revealed a bricked up doorway hence the restraint condition along the respective vertical edge of the inner leaf was a complex mix of door frame connections and continuous URM wall (see *Figure 5*h)

Wall C-2 had one free vertical edge (sawcut through the entire wall thickness) and one URM cross wall support (*Figure 5*i). Both vertical edges of Wall C-3 were made free of restraint by sawcutting (*Figure 5*j).

6. Test setup and instrumentation

A typical in-situ test setup has been shown in Figure 6. Except for wall B-2, the other external walls were tested by applying a uniformly-distributed pressure on the internal leaf by means of a system of airbags. For Wall B-2, airbag pressure was applied from outside the building on the external wall leaf face (Figure 6a).

In all cases timber backing boards were fabricated to the geometry of wall and placed within approximately 100 mm of the wall face being loaded. The backing boards were secured either to the cross walls (where existed) or to the floor. Airbags were placed in the gap between the backing and the wall and inflated to apply pressure on wall surface. The pressure was directly measured using an electronic pressure potentiometer and the data transferred to a data logger.

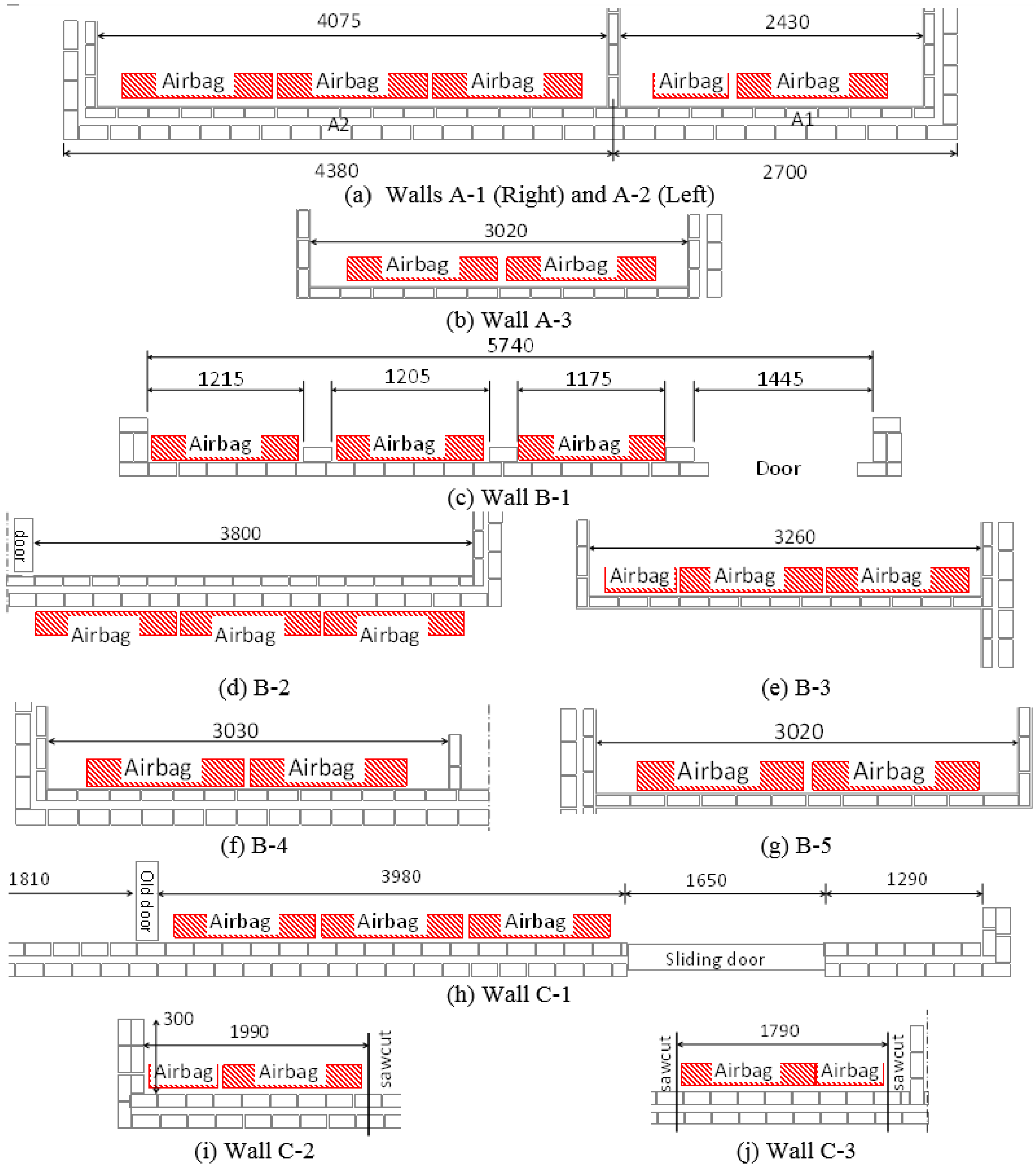


Figure 5: Plan view showing support configuration and direction of airbag loading

Wall displacements were recorded using LVDTs setup (Figure 6b) in accordance with the assumed restraint conditions such that the maximum displacement as well as the horizontal and vertical displacement profile could be captured.

(a) Backing frame (Wall B-2) (b) Typical instrumentation (Wall A-2)

Figure 6: Test setup

7. Observed wall crack pattern

The out-of-plane crack pattern of solid URM walls has been well documented by experimental research [3] although the related tests have been mostly done on specimens built in laboratory and tested under controlled boundary conditions. However, there is no documented experimental research on two-way spanning cavity walls.

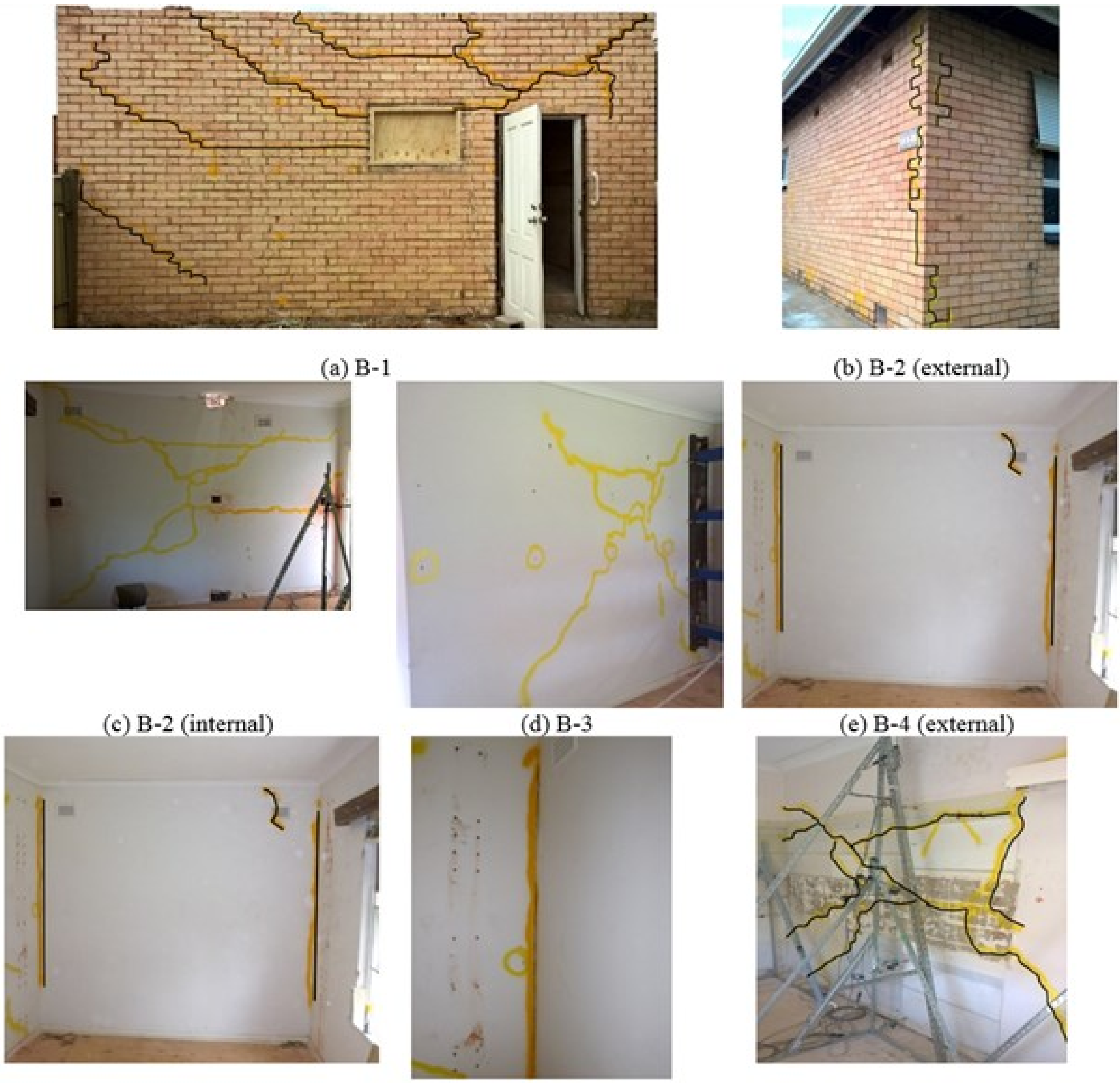
The tested walls underwent cracking (Figure 7 to Figure 9) that was generally consistent with the wall damage pattern previously documented for single-leaf solid walls by other researchers, e.g. [3,4]. Notwithstanding, some details of cracking was affected by existing hairline cracks and mixed boundary conditions that were different from the idealized conditions assumed in laboratory.

(a) External leaf (b) Internal leaf (loaded)

Figure 7: Crack pattern for Wall A-1

The cracking pattern for external leaf of Wall A-1 (Figure 7a) is consistent with that for a wall with lateral restraint along 4 edges, where diagonal cracks extend from the wall corners with an angle that is equal to the natural slope of masonry construction, i.e. height to length ratio of masonry units. This cracking pattern suggests that the wall ties transferred significant amount of force to the external leaf. The damage observed for the internal leaf (Figure 7b) is consistent with a wall supported along three edges, with the top edge being free. A the joint between the top cornice and the wall opened as visible in Figure 7b) due to the little restraint provided by the connection. The observed damage for Wall A-2 was very similar to A-1. The cracking in Wall A-3 confirmed that the roof strut supporting the top of the wall as described earlier herein provided sufficient restraint that prevented deformation at the top boundary. The wall deflected profile will further be discussed in the next sections.

The crack patterns for walls in Building B are shown in Figure 8. The damage in Wall B-1 (Figure 8a) was focused on upper half of the wall due to some out-of-plane slip occurring at the wall ties. The damage included re-opening of the pre-existing cracks in the spandrel above the door opening and the deflected profile (as discussed later herein) suggested that the spandrel provided little restraint, with the wall response being consistent with a three-edge supported wall. Post-test observation of loaded wall face revealed vertical joint between the test wall and the masonry piers indicating no positive connection.



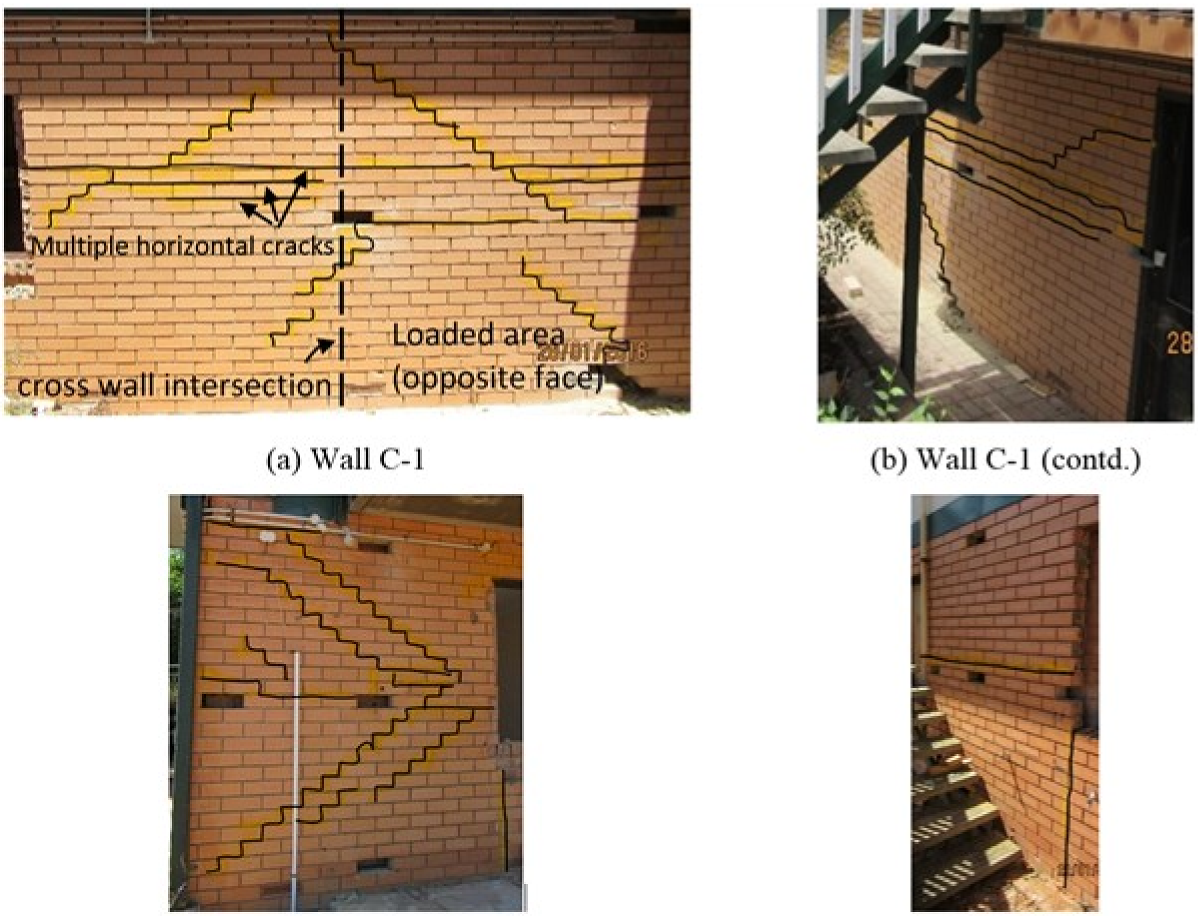
(a) B-1 (b) B-2 (external) (c) B-2 (internal) (d) B-3 (e) B-4 (external) (f) B-4 (internal) (g) B-4 (B-3 corner) (h) B-5

Figure 8: Crack pattern for walls in Building B

The wall that was loaded from external face (B-2, Figure 8b and c) underwent cracking on the internal leaf (Figure 8c) that was slightly shifted upwards from the wall mid-height, similar to the case of Walls A-1 and A-2.

This observation was consistent with the boundary condition at top, which was ceiling cornice with negligible restraint. Some cracking was observed in the adjacent room that suggested the door frame shown in Figure 8c (right vertical edge) did not provide the test wall with substantial restraint. The internal walls B-3 and B-5 (Figure 8d and h) underwent cracking consistent with four-edge supported wall demonstrating the effectiveness of the roof strut connections at top. The crack pattern on the external face of Wall B-4 (Figure 8e) was slightly shifted upwards due to roof flexibility and the lack of rotational restraint along the top edge. The vertical edges of Wall B-4 connection with an external wall and an internal wall (previously tested B-3) showed cracking (Figure 8f) most significantly for the latter (Figure 8g). The larger cracks at this corner with the internal wall was attributed to lack of good interlocking between external and internal walls.

The crack patterns for walls in Building C are shown in Figure 9. The damage on the external leaf of Wall C-1 (Figure 9a and b) suggested primarily one-way vertically distribution of the forces. The horizontal cracks in Figure 9a and its extension in Figure 9b are accompanied by diagonal cracks limited to the central region of the wall. These diagonal cracks were propagated from the intersection of the wall with the internal cross wall. The observed damage in Walls C-2 and C-3 (Figure 9c and d) was consistent with classic damage expected for, respectively, three-edge and two-edge supported walls.



(a) Wall C-1

(b) Wall C-1 (contd.)

(c) Wall C-2

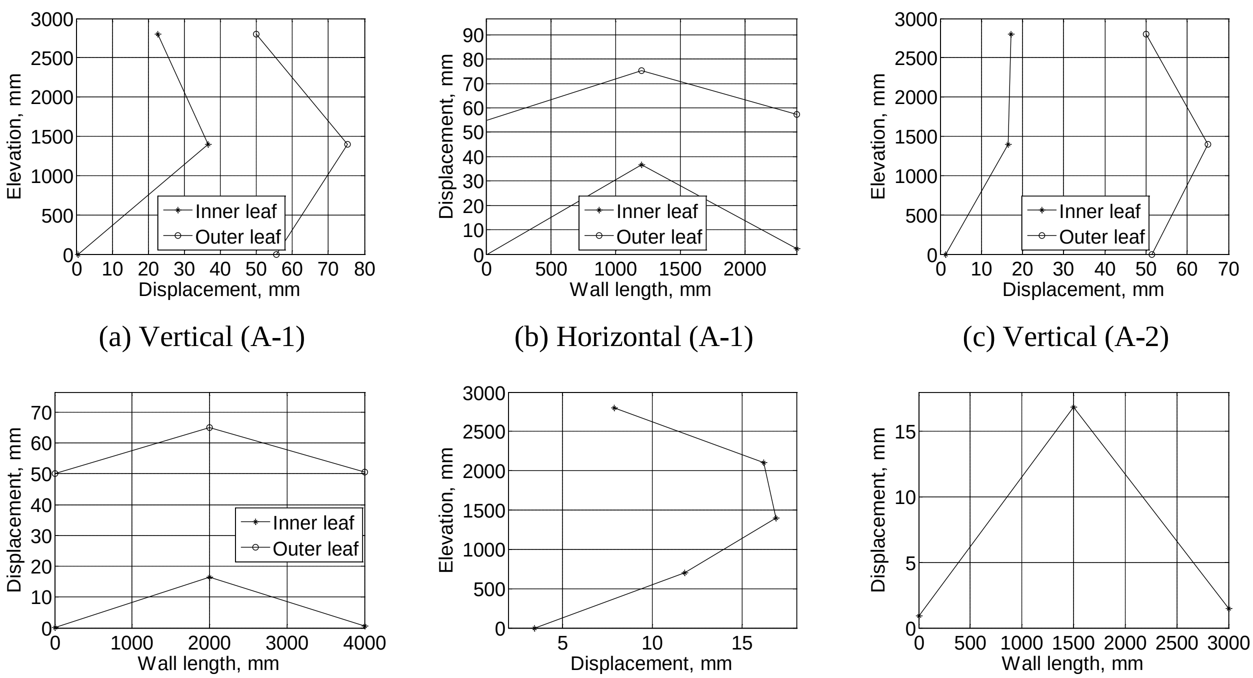
(d) Wall C-3

Figure 9: Crack pattern for walls in Building C

8. Wall deflected profile

The deflected profile of individual leaves was studied both to investigate the capacity of the wall ties to main the gap and to deduce the effects of boundary conditions. The results would be particularly useful in designing a retrofit system for cavity walls. Due to the limited access to the cavity, direction-sensitive surface treatments such as NSM FRP cannot be applied on both faces of the wall leaves. Consequently, each wall leaf can be retrofitted for one direction only. If ties have sufficient compressive capacity, they can be relied upon to transfer the inertia from unretrofitted wall, i.e. analogous to the loaded wall in the tests reported herein, in any direction to the wall leaf that has been retrofitted in that direction.

The wall ties were found to maintain the gap to significant airbag pressures. A typical wall displacement profile has been shown in Figure 10, which belongs to walls in Building A. For brevity, the deflected profile for other walls are not shown but the relevant conclusions are detailed herein. The largest decrease in the cavity width was measured for walls in Building A, with the maximum value being 23 mm and 18 mm recorded at the top centre of, respectively, Walls A-1 (Figure 10a) and A-2 (Figure 10c). The reduction in cavity width was greater at that part of the wall partially due to the shear transfer being maximum at top for the uniform loading that was applied. The other reason was that the top of the inner leaf was unrestrained. The failure mode of the ties was observed to be buckling, i.e. no punching shear was observed. The differential displacement for Wall A-2 was smaller than that for A-1 and the difference was attributed to clogs of mortar that had filled the cavity in two locations, hence assisted in transferring some of the compression forces. The displacement profile for the solid Wall A-3 has been shown to highlight the results of the free boundary condition at top. Similar displacement patterns were observed for walls in Building B that had cavity walls with comparable boundary condition. The wall ties in Building C were observed to maintain the gap for both two-way walls C-1 and C-2 and the one-way vertically spanning wall C-3.



(a) Vertical (A-1) (b) Horizontal (A-1) (c) Vertical (A-2)

(d) Horizontal (A-2) (e) Vertical (A-3) (f) Horizontal (A-3)

Figure 10: Deflected profile of walls in Building A

9. Wall strength

The recorded response of the tested walls has been shown in Figure 11, with the applied airbag pressures being translated into equivalent lateral acceleration in *g* units applied simultaneously to both wall leaves. The estimated ranges of the seismic acceleration applied on a top-storey out-of-plane loaded URM wall in a 5-storey building located on Shallow or Deep soil (values are equal) have also been calculated according to the Australian seismic loading code [8] and shown for regions with different seismicity. The ASCE definition of Low, Moderate, and

High Seismicity [9] was used in Figure 11, with z value corresponding to the Hazard Factor, z, in the Australian seismic loading code [8]. Most of the URM buildings in Australia are located in regions with moderate seismicity, with the estimated seismic demand being well below the recorded strength of the walls.

The wall ties in cavity walls were able to maintain the gap up to airbag pressures represented by equivalent acceleration in Figure 11. This results suggest that the ins-situ cavity ties are able to sustain substantial compressive forces that exceed the seismic demand on walls in regions with moderate seismicity. This finding is useful in the seismic retrofit of cavity walls. One retrofit strategy can be to strengthen each wall leaf in one direction only (as dictated by the limited access to cavity to apply direction-sensitive strengthening methods). The compressive strength of the wall ties can then be relied upon to transfer the inertia of the unstrengthened wall to the retrofitted wall.

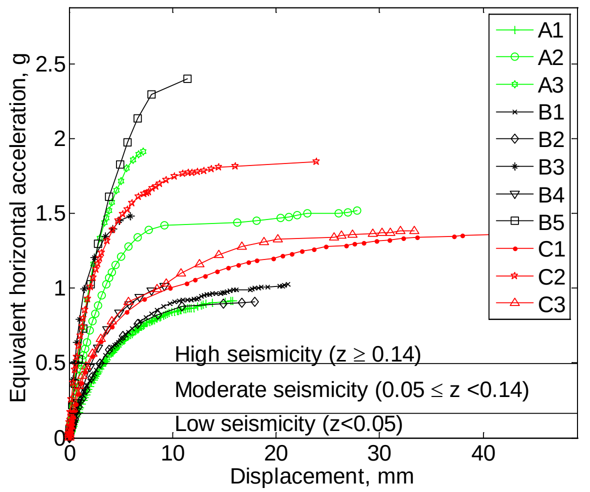


Figure 11: Recorded force-displacement data for all walls and comparison with estimated seismic demand

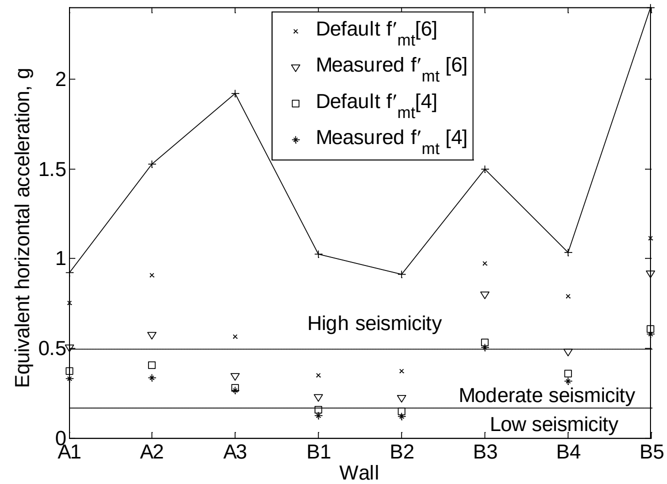


Figure 12: Comparison between the measured and predicted wall strength and the range of seismic demand for regions with different seismicity

9.1 Comparison with code-based evaluation

The Australian Masonry Standards [6] recommends formulae to calculate the out-of-plane strength of two-way spanning URM walls applicable to walls in Buildings A and B. The walls in Building C were subject to arching action that is not represented in the method in [6]. The strength of the tested walls not subjected to arching action was calculated as per the method described in this reference and additionally as per a method developed by [4]. For each method, two different material properties were used one being the default characteristic masonry bond strength, f'_{mt}, recommended as 0.2 MPa in [6], and the other being the characteristic strength obtained using the experimental data from the location of each wall. The experimental characteristic value was calculated as Mean - 1.65 * (Standard Deviation), with a nominal standard deviation equivalent to a Coefficient of Variation of 30% being used due to the lack of sufficient number of data to represent variation for individual walls. In all 4 combinations, a capacity reduction factor of 0.6 was applied to predicted strength as recommended in [6].

The comparison results are shown in Figure 12. The data suggests that notwithstanding the choice of material strength, i.e. measured characteristic vs. default value in [6], the analytical methods significantly underestimated the wall strength. It is highlighted that the measured material strength was greater than default [6] values as also reflected in the wall strength calculations in Figure 12. This comparison indicates that most of the underestimation can be attributed to factors such as wall finish and boundary condition not being as per the idealized conditions assumed in theory, and possible other systematic conservatism in the predictive formulae.

The range of the seismic demands shown in Figure 12 suggests that for the majority of the scenarios the wall would be assessed as having a strength below the code requirements for regions with high seismicity. Most

Australian URM buildings are located in Sydney, Melbourne, and Adelaide, with z ranging from 0.08 to 0.1 (moderate seismicity; refer Figure 11 for range of z values). The results in Figure 12 suggests that the underestimation by the predictive methods may result in many walls in these regions to be deemed unacceptable while in-situ testing may prove their adequate strength.

10. Conclusions

The results of in-situ testing on 10 two-way spanning walls (6 cavity, 4 solid) and 1 vertically spanning cavity wall located in 3 buildings in South Australia were presented. The tests on cavity walls suggested that wall ties had significant compressive strength that resulted in the cavity gap being maintained up to significant strength values that exceeded seismic demand in regions with low to moderate seismicity. The results from the walls not including arching action were compared to analytical predictions and comparisons suggest that the predictive methods may be overly conservative. The measured strength of the walls suggested that two-way walls that are supported along two vertical edges may remain stable in regions with low to moderate seismicity. The calculations of seismic demand on top-storey walls of 5-storey buildings made herein were based on the procedure in the Australian seismic loading codes, and other codes have not been checked in respect to how the ground accelerations should be amplified before being applied to top-storey out-of-plane loaded walls.

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12. References

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