In-plane Testing of URM Wall Panels Retrofitted Using Timber strong-backs

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Abstract

In-plane failure modes are commonly observed in clay brick unreinforced masonry (URM) buildings following strong earthquake-induced ground shaking. To address such potentially weak failure types and to reduce the risk of severe earthquake-induced damage, an experimental in-plane testing programme was undertaken to determine the response behaviour of as-built and retrofitted URM wall panels. In this study in-plane testing of five double-leaf clay brick unreinforced masonry walls was conducted, where the walls were designed to be representative of those generally found in Australia and New Zealand. The walls were tested in as-built and retrofitted conditions using two different methods: (1) timber strong-backs, and (2) plywood overlays. Wall repair attempts were also made using two different techniques: (1) timber strong-backs, and (2) ratchet tie-down straps. The testing procedure and results are highlighted herein. The adopted retrofit solutions both had a positive impact on the seismic performance of the walls.

Keywords: In-plane shear, retrofitted URM walls, diagonal compression test

1 Introduction

The global progression of engineering design and building standards has led to a classification of many existing unreinforced masonry (URM) buildings as structurally inadequate. Of particular concern are URM buildings located in seismic regions, which are highly vulnerable when subjected to strong earthquake-induced shaking. The damage and or partial collapse of these structures is typically attributable to out-of-plane and in-plane failure mechanisms and with the global stability of URM structures being governed by in-plane wall behaviour

(Magenes & Calvi, 1997). When subjected to lateral in-plane loading the URM walls respond in either flexure or shear-controlled failure types, with the flexural mode being referred to as rocking or toe crushing (Dizhur et al., 2013) and the shear mode generally occurring via horizontal bed joint sliding or a diagonal shear mechanism (Dizhur et al., 2013; Ingham & Griffith, 2015). Failure modes such as sliding shear and rocking tend to reduce the displacement capacity of the unreinforced masonry wall, while the comparatively weaker failure mode of cracking can rapidly inhibit the shear capacity of a URM wall component that is subjected to earthquake-induced shaking (Dizhur et al., 2013). The capability of a URM building to withstand seismic loading depends on the type of retrofitting technique used to strengthen the buildings. The structural damage inflicted by earthquakes has prompted the need to review and investigate their effect on existing structures and to propose a retrofitting method that will strengthen both new and existing old URM structures (Shermi & Dubey, 2018)

Many researchers have studied various retrofitting techniques to assess the in-plane behaviour of URM walls over the years. The experimental results of diagonal compression tests of double-wythe masonry panels strengthened with glass fibre reinforced polymers (Kalali & Kabir, 2012) showed the unreinforced specimen experienced brittle failure and a significant increase in load-bearing capacity. In-plane shear improvement of URM walls panels using near surface mounted (NSM) CFRP strips (Dizhur et al., 2013) proved that the adopted retrofitting technique method was simple and inexpensive whilst improving the shear strength and displacement capacity of the damaged URM wall panels. Lin et al., (2014) demonstrated the application of Engineered Cementitious Composite (ECC) to strengthen clay brick unreinforced masonry wallettes and reported that ECC shotcrete enhances the in-plane strength and pseudo-ductility of a URM wall. Ismail and Ingham (2016) and Shabdin et al., (2018) applied polymer textile reinforced mortar to study the in-plane and out-of-plane responses of unreinforced masonry walls. Both studies confirmed that polymer textile reinforced mortar improved the diagonal load carrying capacity and deformation capacity which caused the retrofitted walls to fail in a ductile manner. Shermi and Dubey (2018) researched the in-plane behaviour of unreinforced masonry panels strengthened with welded wire mesh and reported an increase in shear strength resistance and recommended the practical application of their retrofitting technique.

For the repair of seismically damaged URM buildings the use of cost-effective, simple retrofit techniques that will improve the displacement capacity of walls, limit crack propagation and maintain the architectural fabric of the building is essential. Whilst the aforementioned retrofit methods are viable and sustainable, there remains a need to establish retrofitting techniques that can be efficiently implemented post-earthquake events within heritage buildings, in addition to ensuring ample time for the recovery of properties. Recently, Dizhur et al. (2017) tested a retrofit technique that exhibited these characteristics with their application of timber strong-backs to mitigate the out-of-plane failure of URM walls. However, the in-plane performance of the strong-backs was not considered in their scope of research.

The purpose of this study is to assess the retrofit capability of timber strong-backs during the in-plane testing of five double-leaf clay brick masonry walls. All walls tested in the study were composed of materials that enabled the specimens to simulate the expected response of walls in the URM building stock typically found in Australia and New Zealand. The walls were subjected to diagonal compression tests in as-built and retrofitted conditions, the latter of which employed two methods: (1) timber strong-backs and (2) plywood sheets. In addition, repair attempts for two walls were performed and also entailed two methods: (1) timber strong-backs and (2) ratchet tie-down straps (see Fig. 1). The testing apparatus, procedure and results are reported herein.

(a) Use of ratchet ties to secure damaged masonry arch, Cavezzo

(b) Use of timber elements and ratchet ties to secure damaged masonry, L'Aquila Santa Maria di Roio

Figure 1: Repair techniques applied to URM building in Italy (photos by Marta Giaretton)

2 Methodology

Five double-leaf masonry walls were constructed on movable platforms to facilitate easier manoeuvring. The walls were then tested by applying a tension force in a diagonal direction via two loading shoes at opposite corners of the URM walls until failure was observed, with the installation of portal gauges to measure wall displacement under the applied load. The first test that was performed established the as-built wall response, which was to be used as the baseline test for the comparison of retrofitted and/or repaired wall behaviour. The subsequent tests involved the in-plane testing of walls that were strengthened with strong-backs or plywood sheets and/or repaired with ratchet tie-down straps or plywood sheets (see Fig. 2).

2.1 Wall Construction

The dimensions of the masonry walls were 1200 x 1200 mm with a thickness of 230 mm and were constructed by placing a header course of bricks after every third stretcher course. The bricks used for the construction of the walls had been recovered from existing URM buildings. They had approximate dimensions of 230 x 110 x 75 mm (length x thickness x height) and a mean compressive strength of 18.8 MPa (C.o.V = 35%). A mortar mix ratio of 1:2:15 parts of cement, lime and sand was used in order to replicate the mortar composition typically encountered in Australian and New Zealand URM structures. Masonry prisms and mortar cubes were subjected to compressive strength testing (ASTM, 2003a), which produced mean mortar and masonry compressive strengths of 1.7 MPa (C.o.V = 13%) and 8.1 MPa (C.o.V = 37%) respectively. The walls were left to cure for a minimum of 28 days before retrofit installation and testing was undertaken (Dizhur et al., 2013).

Table 1: Properties of tested wall panels

Wall ID [a]	Retrofit type	spacing (mm)	Retrofit screw-tie, Ø & V (mm) [b]
W1	-	-	-
W1-R	90x45mm timber strong-backs	400	Ø-8 V-300
W2	90x45mm timber strong-backs	600	Ø-8 V-300
W2-R	Three rachet-tie downs	300	
W3	90x90mm double timber strong-backs	600	Ø-8 V-300
W4	Plywood sheet 1100x1100x30mm	-	Ø-8 V-300
W5	90x45mm timber strong-backs + plywood sheet (1100x1100x30mm)	600	Ø-8 V-300

[a] R- repaired, [b] Ø – screw diameter; V – vertical spacing

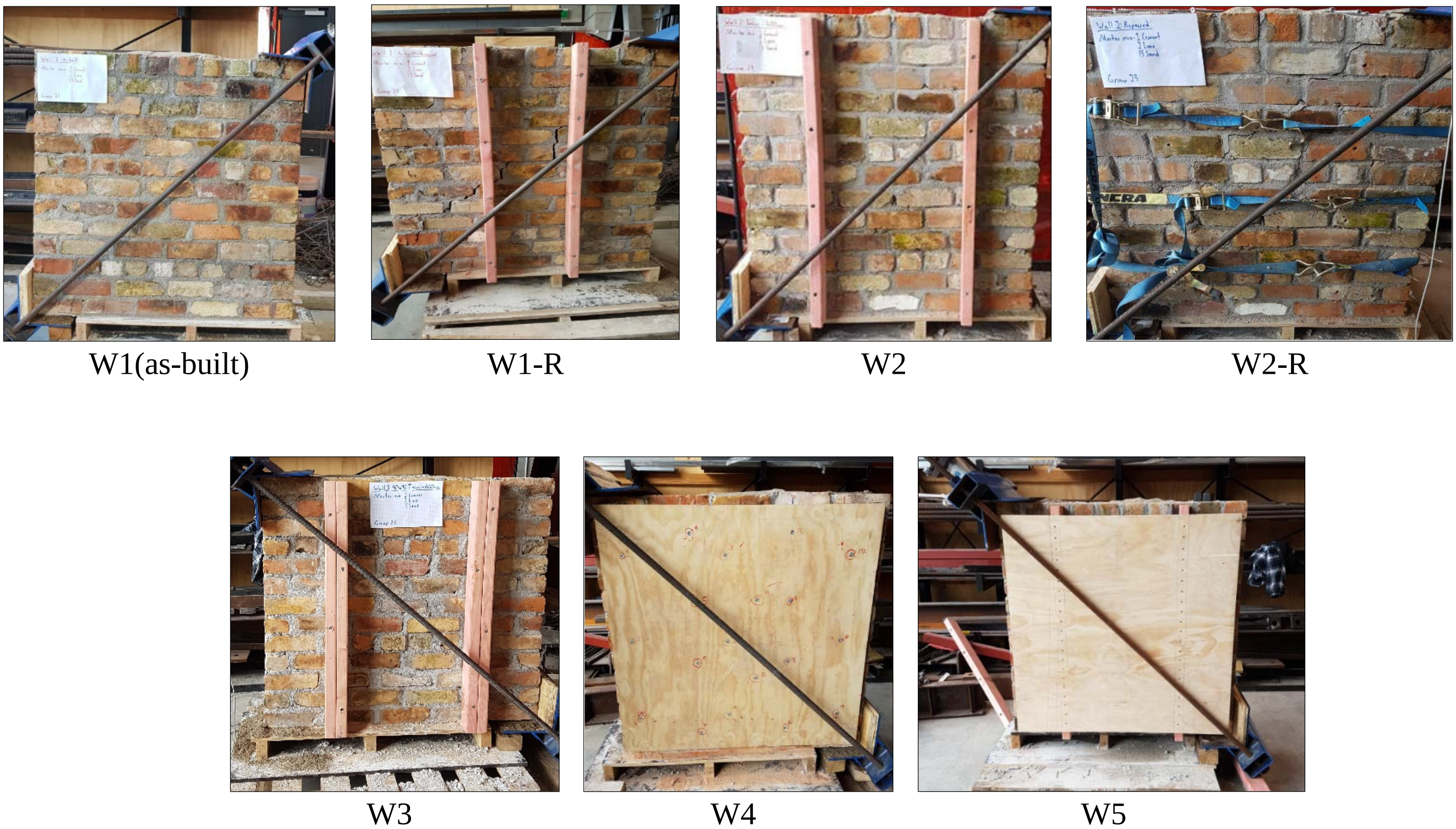
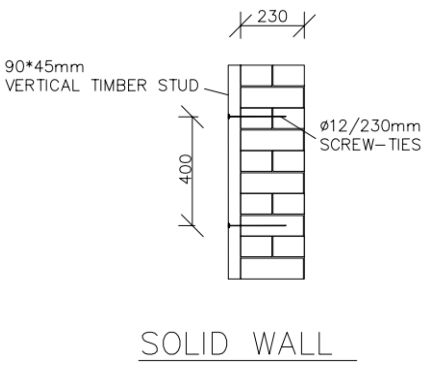


Figure 2: Wall arrangements

2.2 Retrofit Arrangements

The selected retrofitting materials were similar to those used in the test regime conducted by Dizhur et al. (2017), wherein 90 x 45 mm and 90 x 90 mm timber studs were applied to strengthen the URM walls (see Table 1 and Fig. 2**)**. Ø-8 mm diameter screw-ties of 230 mm length and a threaded length of 210 mm were mechanically screwed into the masonry wall through a plywood sheet (1100 x 1100 x 30 mm) and/or timber studs (see Fig. 3**)**. Prior to installation of the screws, pilot holes were drilled through the timber studs and plywood sheets before drilling into the wall with a masonry drill bit to a depth of 200 mm into the brick. The testing of wall W1 in the as-built condition was recorded to establish a reference for comparison with the retrofitted wall tests. After loading, W1-R was repaired using single 90 x 45 mm timber studs at approximately 400 mm horizontal spacing and 300 mm vertical spacing and attached using four Ø-8 mm screw-ties per stud. Wall W2 was retrofitted with the same method utilised for W1-R, however the timber studs were spaced at 600 mm. After removing the timber studs from W2, wall W2-R was repaired with three ratchet-ties spaced at 300 mm. Wall W3 was retrofitted on one face using two 90 x 90 mm timber studs at 600 mm spacing, fixed with Ø-8 mm screw-ties at 300 mm vertical spacing. W4 and W5 were both retrofitted with 1100 x 1100 x 30 mm plywood sheets, however W4 was secured directly on one face of the wall with plywood overlays and W5 was fixed with 90 x 45 mm timber studs spaced at 600 mm. The plywood sheet was secured on both edges of W4 using four Ø-8 mm screw-ties at 300 mm vertical spacing and five Ø-8 mm screw-ties in a staggered pattern at 115 mm from the centre of the plywood. The plywood sheet on wall W5 was screwed on the timber studs at 50 mm spacing in the vertical direction. The screws were placed to enable sufficient shear transfer between the timber strong-backs or the plywood sheet and the URM walls.



(a) Timber strong back connections

(b) Screw-ties

Figure 3: Timber strong back connection and screw

2.3 Test set-up

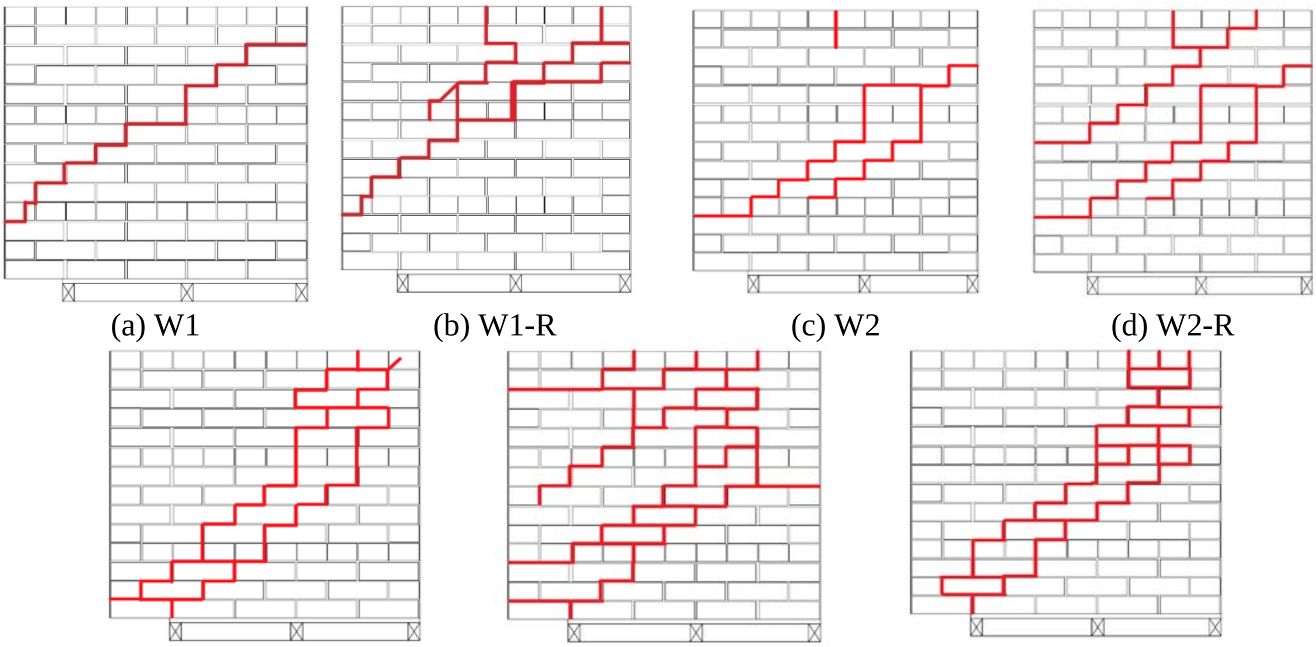
Although testing was undertaken as per ASTM methodology (ASTM, 2007), the testing procedure was altered by performing a 45° rotation of the loading mechanism as opposed to a rotation of the walls. This modification was easily implemented and was performed in order to prevent the risk of crack development which would have an increased potential under rotational manoeuvring of the walls. The installation of the load mechanism involved the placement of two loading shoes at the diagonally opposite corners of the wall (see Fig. 4), which had dimensions of approximately 230 mm length and 350 mm width. The URM walls were initially cantilevered on one side at 230 mm to allow for the placement of one shoe on the bottom corner of the wall. The two loading shoes were connected on both sides of the walls using a high tensile steel rod of 30 mm diameter, which was thus oriented diagonally along the wall. A 300 kN hydraulic actuator positioned between the load cell and the loading plate exerted tension forces on the two steel rods as load was applied, causing the loading shoe located diagonally opposite at the bottom of the wall to exert a compressive force. Load was applied at a uniform rate via a 300 kN hydraulic actuator until an approximate load reduction of 20% in conjunction with visible displacement was observed, thereby indicating failure had occurred. Finally, displacements observed from the two potentiometers attached to the wall were recorded.

Figure 4: Test set up

3 Results and Discussion

3.1 Failure modes and crack patterns

All specimens failed in diagonal shear, with failure typically presenting itself via diagonal cracking along the loading direction. Notably, the crack pattern varied with the retrofitting arrangement that was employed (see Fig. 5).



(a) W1 (b) W1-R (c) W2 (d) W2-R

(e) W3 (f) W4 (g) W5

Figure 5: Crack failure patterns

Wall W1 which represented the as-built response experienced a diagonal shear failure that occurred due to a stepping crack pattern, which generally propagated along the weaker mortar joint. Upon attaining shear stress capacity, a steep diagonal crack had formed which separated the wall into two triangular sections (see Fig. 5a).

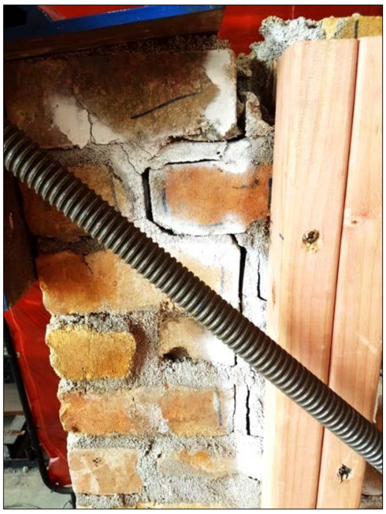
W1-R experienced additional diagonal shear cracks which propagated in the same manner as wall W1 (see Fig. 5b). Additional cracks through the mortar joints were observed at the upper right and left sides of the wall. It is suspected that the new cracks developed due to shear transfer between the timber strong-backs and the unreinforced masonry wall. The existing cracks also grew in size during loading, and the timber strong-backs underwent deformation (see Fig. 6a).

W2 experienced two diagonal cracks with the crack pattern similar to that found in wall W1 (see Fig. 5c). The stiffness of the timber strong-backs limited the displacement capability of the wall and is thus likely to have encouraged the more abrupt dissipation of energy, which was presented in the form of the cracks that indicated diagonal shear failure. The test terminated prematurely, preventing the acquisition of further information regarding the failure mode.

W2-R developed a diagonal shear crack that propagated further than the pre-existing stepped crack in the original wall (W2) (see Fig. 5d). Additionally, the schematic for W2-R indicates some secondary cracks that formed as the wall expanded during loading due to the ratchet-ties preventing the lateral displacement of the wall. In comparison to other test results the size of the cracks was smaller, which is attributed to the elasticity of the ratchet ties. The combination of ratchet tie elasticity and smaller crack size enabled the URM wall to return to its original position after loading.

(a) Wall W1-R response showing timber strong-back deformation

(b) W3 response showing rigid response of double timber strong-backs



(c) Example of tie mobilisation inducing localised masonry deformation in W3 response

(d) Plywood retrofitted W4 response showing induced torsional effect at unrestrained corner

Figure 6: Test observations

Wall W3 exhibited two diagonal cracks that ran parallel to the line of action of the applied load (see Fig. 5e). The amount of cracking significantly increased as the applied load increased which was attributed to the double timber strong-backs inducing greater wall rigidity, and it was observed to concentrate around the Ø-8 mm screw ties connecting the strong-backs to the wall (see Fig. 6c). The double timber strong-backs remained rigid throughout the test compared to the single member strong-backs in wall W1-R (see Fig. 6b). Due to the restriction of lateral displacement, the wall gradually lifted at its unrestrained corner as it attained shear stress capacity.

The plywood retrofit wall W4 developed the largest proportion of cracks which ran in the diagonal loading direction (see Fig. 5f). Due to an increased shear transfer, the ultimate shear stress of the wall contributed to the development of the diagonal crack. Toe crushing was observed at the lower corner of the wall where the bond between mortar and brick was lost. There was also a torsional effect at the unrestrained upper corner of the wall (see Fig. 6d) which was due to a pulling force that buckled the plywood, promoting shear sliding and the formation of a diagonal shear crack.

W5 underwent a diagonal shear crack pattern similar to W3 (see Fig. 5g) and exhibited toe crushing at the lower corner of the wall where the bond between mortar and brick was lost. The wall also experienced a torsional effect, albeit less significant compared to W4. Timber strong-backs and plywood overlays remained rigid, and as a result there was a lift at the unrestrained corner of the wall.

3.2 Wall Behaviour

Equations 1 and 2 (Lin et al., 2014) were used to calculate the shear stress, τ, and horizontal drift, δ.

Table 2. Results

Wall	**P_{max} kN**	**F_{max} kN**	**τ_{max} MPa**	**δ_{max} %**
W1	61.00	43.00	0.16	1.01
W1-R	40.70	28.75	0.10	5.46
W2	54.35	38.27	0.14	2.05
W2-R	34.69	24.50	0.09	1.05
W3	40.54	28.42	0.10	6.39
W4	61.94	24.02	0.16	6.07
W5	54.00	37.89	0.14	9.17

$$\tau = \frac{P cos\alpha}{0.5t(H+B)} \quad (1)$$

$$\delta = \frac{\Delta S + \Delta L}{2g}(tan\alpha + cot\alpha) \quad (2)$$

where,
P is the measured diagonal compression force.

α is the tan inverse of H/B
H and B are the height and width of the wall
t is the thickness of the wall
δ is the horizontal drift
ΔS is the distance of the wall shortened due to the potentiometers
ΔL is the distance of the wall that lengthened due to the potentiometers
g is the gauge length

Figure 7 shows the shear stress against the drift percentage of the tested walls whilst Table 2 provides a summary of the results. Due to the small number of specimens that were tested within the experiment, it is not statistically possible to establish a relationship by correlating the forces attained with the shear stresses experienced. However, all retrofitted walls displayed prolonged shear capacity following crack development under unreinforced conditions by allowing a greater amount of drift to occur.

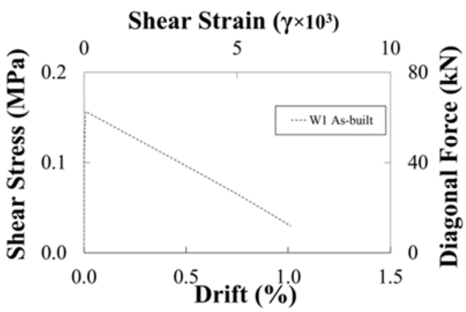
As shown by the graph of the as-built wall (W1), the wall displayed the expected rigid behaviour by attaining shear capacity with minimal displacement before a loss of shear stress was observed (see Fig. 7a). The formation of cracks led to a decrease in the compression force, which reached a maximum value of 61 kN at a drift of 1.01%.

It was unexpected that the maximum diagonal force and shear stress of wall W1-R dropped when compared to the as-built wall W1. Despite a significant increase in drift W1-R did not display an improvement in shear stress, which is likely attributed to the timber strong-backs that were used for repair encouraging a prolonged shear resistance (see Fig. 7b).

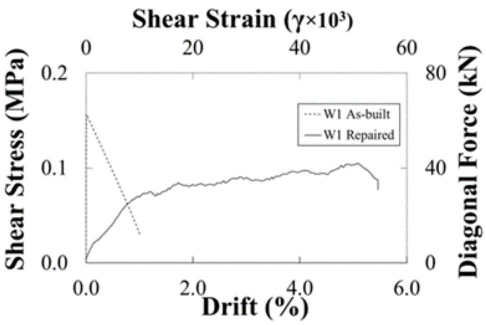
Walls W2 and W2-R shared a similarity in displaying a smooth curve relationship between shear stress and drift which indicates ductility. However, during the experiment of wall W2 the software stopped functioning which prematurely ended the test and prevented the formation of cracks in addition to the double diagonal shear cracks that had propagated. It is therefore probable that this malfunction affected the results produced from the testing of these walls (see Fig. 7c and d).

The graph of wall W3 indicates that upon reaching a shear stress capacity of 0.1MPa, the double timber strong-backs enabled a prolonged shear resistance by allowing an increased percentage of drift. The wall achieved a maximum drift of 6.4% whilst maintaining some resistance to the applied load before the test was concluded.

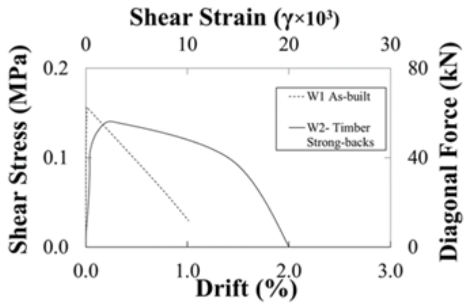
The graphs of walls W4 and W5 indicated a behavioural similarity with wall W3 by demonstrating a ductile failure response for both specimens, wherein both walls maintained constant shear stress values as the drifts significantly increased. The presence of plywood on the two walls may have contributed to these higher drift values (see Fig. 7f & g).



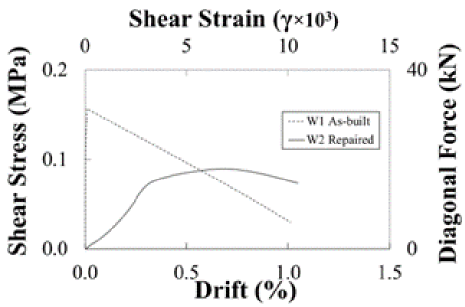
a) W1



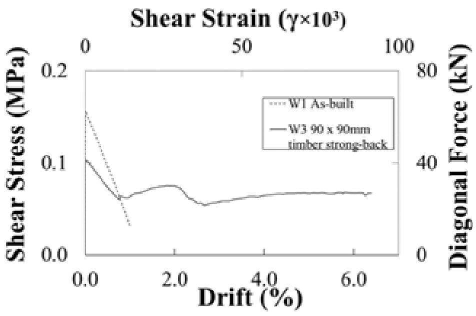
b) W1-R



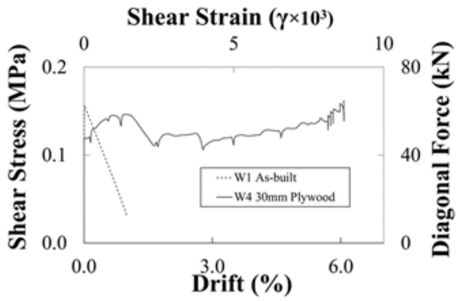
c) W2



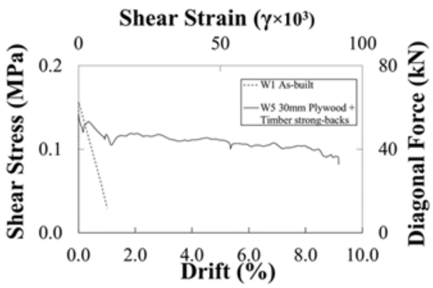
a) W2-R



b) W3



c) W4



g) W5

Figure 7: Wall responses

4 Conclusions

To examine the effectiveness of the retrofit and repair techniques proposed against in-plane failure, five URM walls were tested in diagonal compression. In particular, the following conclusions were drawn:

All specimens failed in diagonal shear, which was predominantly exhibited in the form of diagonal shear cracks through the mortar joints along the line of action of the applied load.

- Wall panels which were retrofitted with plywood sheets were able to resist the diagonal tension force compared to those with timber strong-backs. The retrofitted and repaired walls maintained a reasonably constant shear stress capacity following crack formation.

- The drifts from the retrofitted walls (except W2 and W2-R) indicated a high ductile behaviour of the walls. The screw-tie connections that were used displayed a high buckling resistance due to the formation of cracks around the ties without any tie deformation. This indicates that they contributed in the transfer of applied load to the retrofit material.

- It is imperative to investigate the torsional effect at the unrestrained upper corner of the wall due to the pulling force that buckles the plywood sheet. For the specimens W2 and W2-R which terminated halfway during testing, it is worthwhile to repeat the testing procedure to ascertain the full in-plane response.

- The use of timber strong-backs or ratchet tie-down straps was shown to be an effective method to improve wall in-plane capacity.

- This testing programme was a proof of concept and further testing will be undertaken in the future to incorporate more test parameters.

Acknowledgment

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