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SECURING SOLUTIONS FOR FACE-LOADED CLAY BRICK URM WALLS

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ABSTRACT

Out-of-plane failures induced by earthquake loads are one of the most critical deficiencies of clay brick unreinforced masonry (URM) buildings. Despite a number of seismic improvement techniques having been previously investigated and applied, there is a significant lack of experimentally validated solutions that consider the viability of these interventions in terms of overall associated cost and practicality, and impact on the building tenants, aesthetics and heritage building fabric. The main objectives of the research presented herein were to develop and validate seismic securing techniques for URM walls that satisfied the above conditions, in consultation with industry representatives. Shake-table testing of three full-scale double-leaf solid clay brick URM walls was undertaken. Wall specimens were H3300 × W1200 × T220 mm and closely simulated in-situ conditions. The vertical timber framing that is typically a non-structural support of the inner wall lining was used as part of the retrofit solution and was fixed to the wall with steel brackets and mechanical screw-ties in order to form a strong-back. Post-tensioning was also investigated as a second form of retrofit intervention. Wall and retrofit construction details, test set-up, observed crack-patterns, peak ground acceleration (PGA), wall acceleration and displacement profiles at failure, and quantification of the improvement in seismic capacity associated with use of the proposed retrofit techniques are presented herein.

KEYWORDS: *out-of-plane, masonry retrofit, seismic improvement, shake-table, strong-back, clay brick URM wall*

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INTRODUCTION

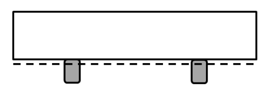
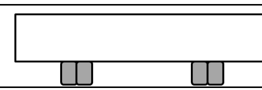
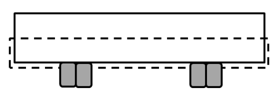
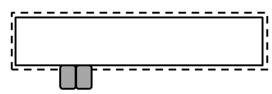
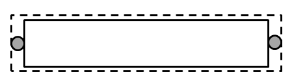
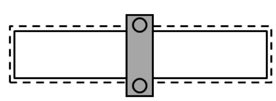
Clay brick unreinforced masonry (URM) has historically been used around the world as a popular construction material for loadbearing applications. Over the last century the inherent weakness of URM walls in their out-of-plane direction when subjected to an earthquake induced lateral load was highlighted during a number of earthquakes [1][2][3][4][5]. Along with non-structural URM elements such as chimneys and parapets, out-of-plane collapse of URM walls pose a significant risk to building occupants and pedestrians during and after an earthquake. Observations from earthquake damaged URM buildings indicated that the mechanism governing out-of-plane failure was either cantilever, one-way bending, or two-way bending failure depending on the boundary conditions of the wall [4]. In order to address and mitigate this risk, simple and cost effective seismic securing solutions are required.

The experimental shake-table campaign reported herein was conceived to investigate the performance of two-leaf-thick solid clay brick URM walls retrofitted using different configurations of timber strong-backs and of tensioned threaded steel rods. A strong-back system is a secondary system, such as a frame, with the primary function of providing out-of-plane support for an unreinforced or under-reinforced masonry wall. A strong-back system is designed and connected to the wall so that the wall develops the full strength of the strong-back, with the wall also required to be fixed to the diaphragms in order to provide a robust load path into the diaphragm. A similar approach was proposed by King et al. [6] using steel strong-backs as a retrofit strategy to protect masonry cladding structures from blast loading. The choice of using timber as a retrofit material comes from investigations of existing URM buildings in New Zealand that showed that a large portion of these buildings had timber framing lined with plasterboard as the interior finish. Hence, validating a securing solution that connected the masonry to the timber framing as a load path into the diaphragm would provide a practical and low-cost seismic securing method. The concept is also analogous to the timber framing used as an earthquake-resistant system for masonry buildings during the Minoan era [7] and later extended to the entire Mediterranean area [8]. The other retrofit technique investigated herein was use of the post-tensioning technique which was observed to perform well during the 2010/2011 Canterbury earthquakes [9]. Experimental testing of unbonded post-tensioning also showed increased strength of historic clay brick masonry when subject to pseudo-static reverse cyclic out-of-plane loading [10].

EXPERIMENTAL PROGRAMME

Three two-leaf-thick solid clay brick URM wall panels (3300H × 1200W × 230T mm) were subjected to dynamic out-of-plane loads using a shake-table. The height of the tested panel was 3000 mm up to the diaphragm level with an additional 300 mm high parapet, with the aim being to replicate the traditional top-storey perimeter wall of an URM building [11]. The first wall was initially tested using 90 × 90 mm timber strong-backs applied in three different configurations: (i) 90SB, wall strong-backs and as-built parapet, (ii) 90SB-p1, wall strong-backs and parapet secured only on the top and inner side (version V1), (iii) 90SB-p2, eccentric wall strong-back and parapet secured on the top and both inner and outer sides (version V2). Lastly the first wall was tested in the as-built condition, URM-p, with the parapet being secured (version V2) in order to identify the response of the wall and avoid premature failure of the URM parapet. The second wall was tested in the retrofitted condition by applying two 90 × 45 mm timber strong-backs on the inner side and securing the parapet on the inner side with strong-backs and mechanical screws (version V3), 45SB-p. The third wall was tested using two different post-tensioning solutions: (i) 20PTi-p, applied within the wall cross-section using two Ø20 mm steel threaded rods either side, and (ii) 12PTe-p, applied externally using two Ø12 mm steel threaded rods positioned front and back at the centre of the wall. Table 1 shows the text matrix including schematics and photos of each retrofit configuration.

Table 1: Solid wall test matrix (refer to Figure 3 for the types of parapet securing)

Wall	Securing Type	Spacing (mm)	Schematic	Photo example
URM-p	URM + parapet securing V2	-		
45SB-p	2 x (90 x 45 mm) timber strong-back + parapet securing V3	600		
90SB	2 x (90 x 90 mm) timber strong-back (URM parapet)	600		
90SB-p1	2 x (90 x 90 mm)timber strong-back + parapet securing V1	600		
90SB-p2	1 x (90 x 90 mm) timber strong-back + parapet securing V2	1200		
20PTi-p	2 x Ø20 mm post-tensioned steel threaded rod internally cored wall + parapet securing V4	1200		
12PTe-p	2 x Ø12 mm post-tensioned steel threaded rod applied externally + parapet securing V5	1200		

Wall construction

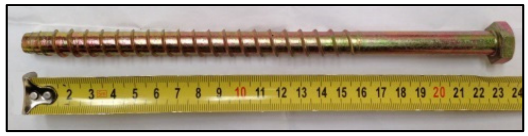
Test walls were constructed using a common brick pattern with mortar joint thickness of approximately 10–15 mm and recycled (230L x 110W x 75H mm) clay bricks obtained from demolished vintage URM buildings constructed in the 1920s/1930s. As there is significant variability in brick properties within an existing vintage building, the reuse of vintage bricks introduced realistic material variability into the tests. The mortar mix was made from sand and lime in the ratio of 3:1 by volume respectively, with the intent of replicating the weak mortar condition typically encountered in historic New Zealand URM buildings. 50 ×50 ×50 mm mortar test cubes were prepared during wall construction and tested in compression after 28 days to obtain an average compressive strength of 0.54 MPa [12]. The mean compressive strength of individual bricks, estimated using the half brick compression test [13], was 30.5 MPa, while the mean compressive strength of masonry prisms was 8.2 MPa in accordance with [14]. To facilitate the relocation and positioning of wall samples onto the shake-table, the walls were constructed with 8 mm steel channels top and bottom tied at the edges with threaded rods.

Mitigation solutions

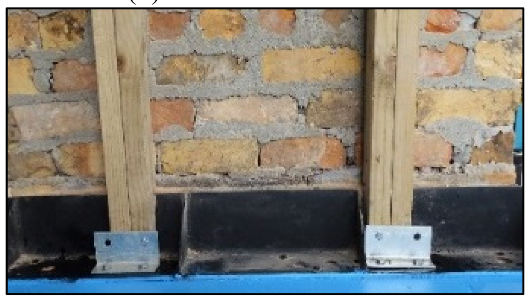
Based on previous testing of solid clay brick masonry cavity-walls retrofitted using timber strong-backs [15], 90 x 45 mm standard timber studs were secured to the masonry using Ø12/230L mm mechanical screws (pull-out capacity of 18 kN), as shown in Figure 1. The first application consisted of single 90 x 45 mm timber studs applied with 600 mm spacing, 45SB-p (see Table 1). The second application consisted of double 90 x 45 mm timber studs (corresponding to an equivalent 90 x 90 mm timber stud) applied with 600 mm spacing in wall samples 90SB and 90SB-p1, and installed with 1200 mm spacing in 90SB-p2 (see Table 1). The mechanical screws were installed with a spanner in pre-drilled Ø12 mm holes and were located at the centre of the timber studs with a vertical spacing of approximately 500 mm. The masonry was drilled using an impact-drill, making sure to limit vibrations in the wall. The mechanical screws and washers were countersunk 10 mm into the timber strong-backs to provide a smooth surface for wall linings, see Figure 1a. The base of the strong-backs was fixed to the shake-table using a 5 mm thick steel bracket and two Ø12 mm standard timber screws, to allow the shear induced in the strong-back to be transferred to the table/ground, see Figure 1c. The top of the strong-back was fixed to the roof-diaphragm using steel brackets and 30 mm long Ø5.5 mm standard timber screws. Standard GIB plasterboard was fixed to the timber strong-backs to demonstrate the aesthetic finish achievable using the securing technique, see Figure 1d.

Post-tensioning was installed using two different configurations (Table 1). Wall 20PTi-p was secured with two Ø20 mm steel threaded rods located at the centre of the wall cross-section at the edges of the wall panel, see Figure 2a. In practice, the wall would have to be vertically cored using specialist tools so that the rod could be fixed to the base of the building, which would be expensive and labour intensive. The steel threaded rods were connected to an 8 mm thick steel plate placed along the width of the base and the top of the wall.

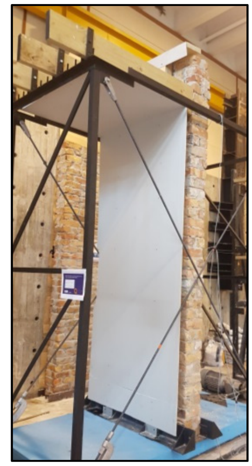
(a) Timber strong-backs



(b) Mechanical screws



(c) 5 mm steel brackets fixing base of strong-backs



(d) Plasterboard finish

Figure 1: Example of installation process of retrofit timber strong-backs

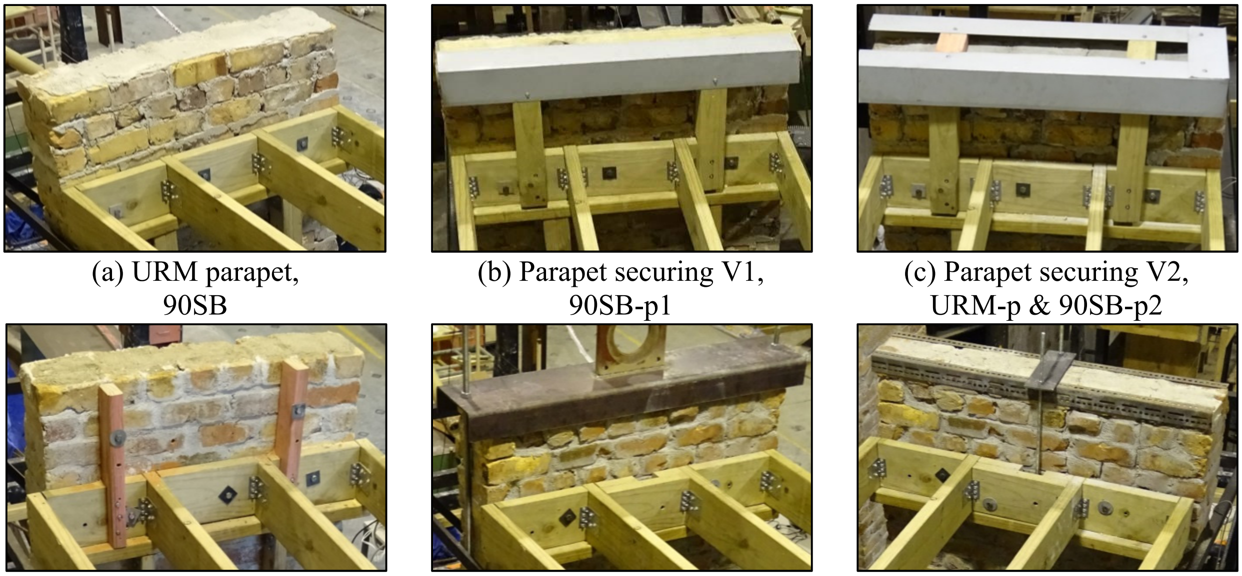
(a) Ø20 mm post-tensioned steel rod simulating internally cored wall

(b) Ø12 mm post-tensioned steel rod applied externally

Figure 2: Types of post-tensioning tested (highlighted with white line)

Wall 12PTe-p was retrofitted using Ø12 mm steel threaded rods located externally at the centre of both front and back wall sides (see Figure 2b) with the aim to reduce the installation costs. Being exposed on the exterior face of the wall, the steel rods needed to be as minimally visually intrusive as possible and hence a smaller diameter in comparison to the 20PTi-p securing solution was used. The steel threaded rods were secured to the base plate by welded M20 couplers and secured at the top with a nut. The 20PTi-p and 12PTe-p were tensioned using a common hand spanner. The reason for minimising the tension on the steel rods was to simulate a post-tensioned retrofit where the tension in the steel had relaxed, replicating a worst case scenario [10].

All tested wall panels had a 300 mm high URM parapet above the roof-diaphragm which typically reproduced the type of retrofit technique adopted for the wall below. Figure 3 shows all the different versions of parapet securing adopted during testing, including the unreinforced parapet (see Figure 3a) tested in wall 90SB.



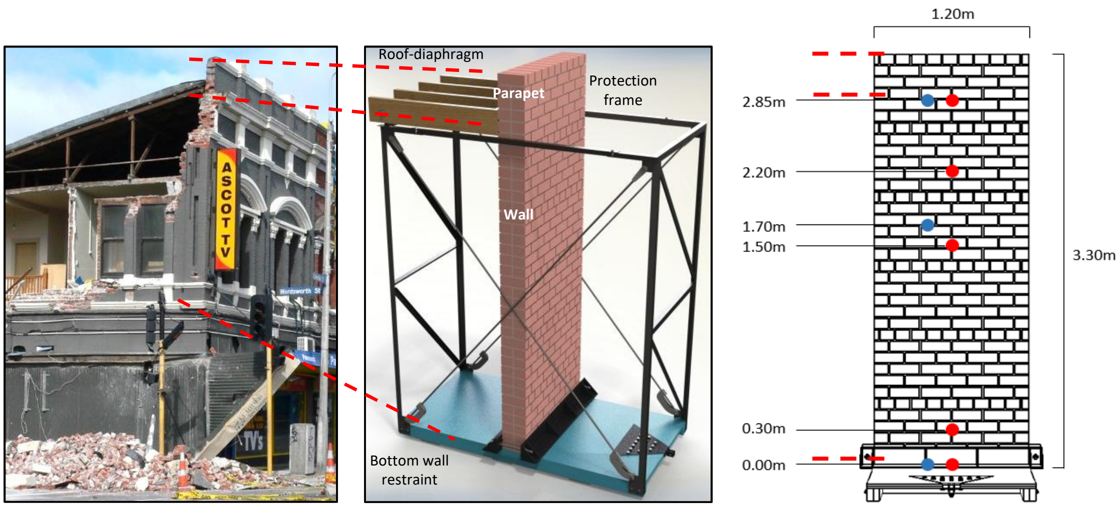
(a) URM parapet, 90SB
(b) Parapet securing V1, 90SB-p1
(c) Parapet securing V2, URM-p & 90SB-p2
(d) Parapet securing V3, 45SB-p
(e) Parapet securing V4, 20PTi-p
(f) Parapet securing V5, 12PTe-p

Figure 3: Types of parapet securing adopted

Three versions (V1 to V3) of strong-backs were installed on four parapets (walls URM-p, 90SB-p1, 90SB-p2, 45SB-p). V1 consisted of 90 x 90 mm @ 600 mm spacing timber studs fixed at the roof-diaphragm with a single-side horizontal restraint at the top of the parapet to avoid overturning, see Figure 3b. V2 was a further improvement of the first version, which included the horizontal restraint at the top of the parapet for both inner and outer parapet sides, see Figure 3c. Lastly, in V3 the horizontal top restraint was removed and Ø12/230L mm mechanical screws were used to fix the 45 x 90 mm @ 600 mm spacing timber studs directly to the masonry, see Figure 3d. Post-tensioning was adopted for two parapets (walls 20PTi-p and 12PTe-p, versions V4 and V5) by simply extending the retrofit technique used for the wall up into the parapet, with steel threaded rods and a horizontal top restraint as shown in Figure 3e-f.

Test set-up

The test set-up was designed to mimic in-situ wall boundary conditions as shown in Figure 4a-b. The base of the wall panel was secured with strong mortar between two stiff steel angles to prevent lateral movements of the wall base. The timber diaphragm was installed at 2900 mm up the wall height and anchored to the wall using Ø12/230L mm mechanical screws and 50 mm square washers to ensure adequate bearing between screw head and timber. Four 1500 mm lengths of 190 x 45 mm timber were used to replicate the roof diaphragm and were fixed to the wall at 300 mm centres using steel brackets and Ø30 mm screws. The roof diaphragm joists were then pin connected to the surrounding protection frame using 5 mm thick steel brackets and Ø12 mm high tensile bolts. The hinge restraint allowed free rotation and uplift at the top of the wall during testing. Above the roof joists was a 300 mm URM parapet. The protection frame was fabricated using 50 x 50 x 5 mm equal angles with shop welds at the connections, and was fixed directly onto the shake-table. The frame was braced using steel bars and steel braces to withstand the load transfer from the wall into the protection frame.



(a) Damaged building in 2010/2011 Canterbury earthquakes

(b) Test set-up

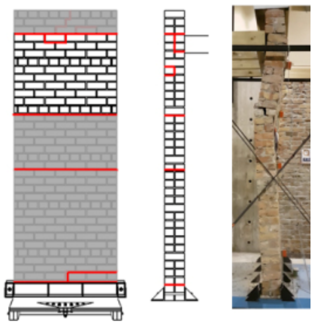
(c) Instrument locations (red dots represent accelerometers and blue are displacement measuring devises)

Figure 4: Test simulation of in-situ URM wall

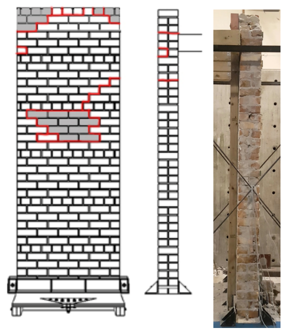
Four accelerometers were fixed to the exterior face of the wall at the base, mid-height, three quarter-height and top, and a fifth accelerometer was installed onto the shake-table in order to record the effective horizontal acceleration produced (see Figure 4c). Two string potentiometers were mounted at the top and mid-height of the wall, as shown in Figure 4c, to measure the differential displacement of the panel. A single-axis acceleration-controlled sinusoidal test transitioning from 0.5 Hz to 50 Hz was applied with increasing acceleration of approximately 0.05g every 15 seconds and constant amplitude at 50 mm. All walls were tested until displaying signs of instability and within the range of the maximum possible load generated by the shake-table.

TEST RESULTS

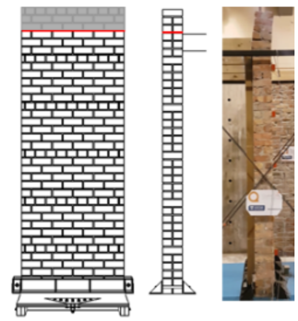
The results are presented herein in terms of damage pattern and failure modes (see Figure 5), Peak Ground Acceleration (PGA), and maximum mid-height and top displacements measured at the point of wall instability (see Figure 6). The acceleration and displacement profiles shown in Figure 6 are useful tools to compare the response during testing of each retrofit solution installed with the as-built condition, allowing consideration of their effectiveness. A summary of the results in relation to all the aforementioned factors is presented in Table 2.



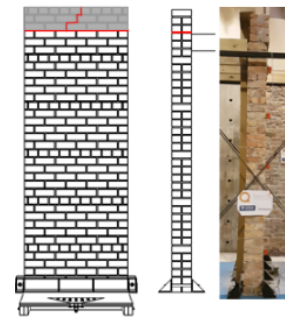
(a) URM-p



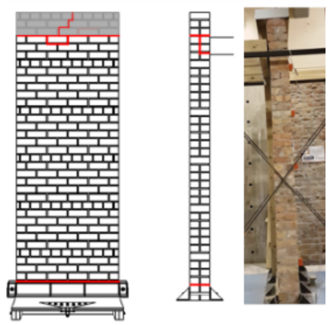
(b) 45SB-p



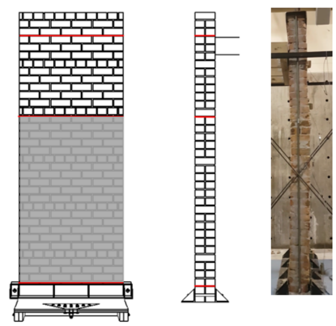
(c) 90SB



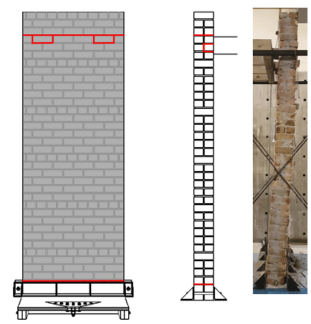
(d) 90SB-p1



(e) 90SB-p2



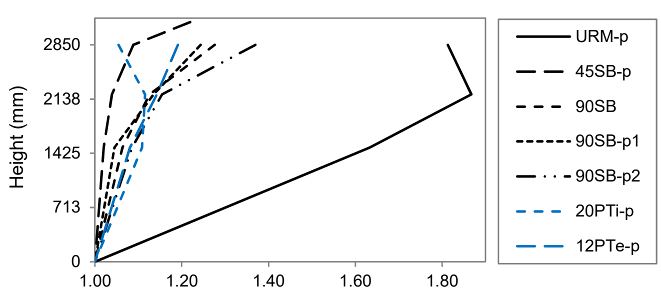
(f) 20PTi-p



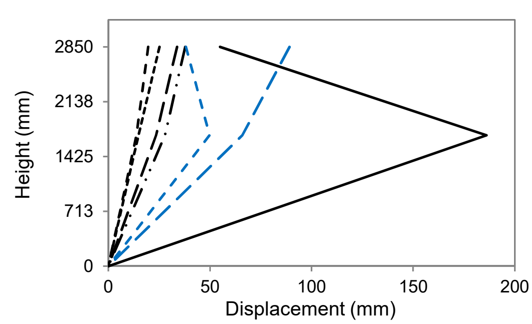
(g) 12PTe-p

Figure 5: Screenshots showing crack-pattern survey and failure progression. Cracks are marked in red and collapsed areas are shaded grey

The wall tested in the as-built condition, URM-p, displayed a typical one-way bending out-of-plane failure with major cracking at three quarter-height and minor cracking at mid-height (see Figure 5a and Figure 6b). The formation of the three quarter-height crack caused a hinge effect, with the wall parts above and below beginning to rock as two separate almost rigid bodies as shown in Figure 5a, inducing a large increase in acceleration at this level (see Figure 6a). As the three quarter-height displacement increased, the flexural capacity of the wall was exceeded, causing the wall to collapse at 0.46g. The maximum displacement recorded near-collapse was 55 mm at top and 186 mm at mid-height (see Figure 6b).



(a) Acceleration amplification



(b) Displacement

Figure 6: Acceleration and displacement profiles at wall instability

45SB-p was retrofitted using 90 x 45 mm timber strong-backs from wall base to parapet top with mechanical screws installed also in the parapet. The first damage observed was the cracking and consequent falling of bricks at the parapet edges, external to the strong-backs, as shown in Figure

5b. With increasing motion intensity a flexural behaviour was observed, leading to crack formation at three quarter-height as occurred for the as-built condition, but in this case the crack was followed by bricks being expelled from the surrounding area which involved only the outer leaf (see Figure 5b). The acceleration reached 1.33g, which was three times higher than that for the as-built condition, and increased linearly up the wall height with a sudden large increase recorded in the parapet, see Figure 6a. The displacement profile was also linear, with 34 mm being recorded at top and 23 mm at mid-height, see Figure 6b, corresponding to a reduction with respect to the as built condition of 39% and 87% respectively.

Table 2: Summary of results

Wall ID	PGA		Max mid-height displacement		Max top displacement		Failure mode
URM-p	0.46 g	(-)	186 mm	(-)	55 mm	(-)	One-way bending
45SB-p	1.33 g	(289%)	23 mm	(13%)	34 mm	(61%)	Flexural behaviour
90SB	0.95 g	(205%)	14 mm	(7%)	20 mm	(36%)	Rigid body behaviour and parapet rocking
90SB-p1	0.97 g	(209%)	15 mm	(8%)	25 mm	(46%)	Rigid body behaviour and parapet sliding
90SB-p2	0.82 g	(177%)	28 mm	(15%)	38 mm	(69%)	Rigid body and torsional behaviour
20PTi-p	0.75 g	(163%)	50 mm	(27%)	38 mm	(69%)	One-way bending and parapet cracking
12PTe-p	0.85 g	(184%)	66 mm	(35%)	89 mm	(162%)	Roof-diaphragm detachment and cantilever
(%) comparison to the as-built value							

Walls 90SB and 90SB-p1 were both retrofitted using 90 x 90 mm timber strong-backs, with the parapet being un-retrofitted in 90SB and being retrofitted in 90SB-p1. The linear displacement profile in Figure 6b clearly shows that the 90 x 90 mm timber strong-backs significantly increased the wall monolithic behaviour and prevented any cracks from forming. In wall 90SB the un-retrofitted parapet exhibited rigid-body rocking behaviour after cracking formed at the roof diaphragm level (parapet base), see Figure 5c. In wall 90SB-p1 the parapet was retrofitted with strong-backs and a single-side horizontal top restraint (V1), preventing rocking failure but allowing the parapet to slide outwards on the existing cracking plane as motion intensity increased, see Figure 5d. 90SB and 90SB-p1 behaved similarly in terms of acceleration and displacement along the wall height (see profiles in Figure 6). Instability due to parapet rocking or sliding was reached at approximately 0.96g (average value), corresponding to a maximum displacement of approximately 15 mm at mid-height and 23 mm at top. The recorded PGA was twice the value reached in the as-built condition and the reduction in displacement was 85% at mid-height and 77% at top. In wall 90SB-p2 the eccentricity caused by the strong-back position increased the stiffness of one end of the wall configuration in comparison to the other end, resulting in the initiation of torsion. A crack formed at the wall base, starting from the side without strong-back and eventually propagated all the way through the base as the shake-table accelerations increased, see Figure 5e. The crack at the base allowed rocking to develop in the whole wall, which led to an increase in the displacement at the roof diaphragm level. The ultra-weak mortar did not provide enough friction against the increasing displacement, enabling brick pull-out where the mechanical screws were tied and the formation of a 15 mm gap between the wall and the roof diaphragm. Consequently the displacements registered were approximately twice those experienced by 90SB and 90SB-p1, even though the PGA was lower (0.82g, see Table 2). The single strong-back provided a sufficient increase in stiffness to prevent any cracks developing at the three quarter-height and mid-height, hence providing securing from out-of-plane failure. The parapet had a double-sided horizontal top restraint (V2) and hence did not present further damage.

The use of post-tensioning in 20PTi-p and 12PTe-p increased the compression loading sufficiently to prevent any cracks forming at low acceleration intensities. As the table amplitude increased, a flexural behaviour was observed that caused the ultra-weak mortar to settle/compact, leading to reduced tension in the steel rods. Because of the interaction with the roof diaphragm, cracking first formed at the parapet base in wall 20PTi-p and was then followed by cracks forming at the base and at the three quarter-height due to the one-way bending flexural behaviour (see Figure 5f and Figure 6b). Instability occurred at 0.75g (63% higher than for the as-built condition but lower than for the other retrofit system tested, see Table 2), with the largest acceleration being recorded at the three-quarter height (see Figure 6a). 50 mm and 38 mm were the displacements achieved at mid-height and top respectively. 12PTe-p initially acted as a rigid body until the crack at the parapet base propagated, causing brick pull-out where the roof diaphragm was connected to the wall, resulting in cantilever behaviour (see Figure 5g and Figure 6b). Wall instability occurred at 0.85g, exhibiting large displacement at the top (89 mm) due to detachment of the roof diaphragm. The mechanical screws proved to have sufficiently pull-out strength to withstand the dynamic loading. Bricks were also observed to be hanging from diaphragms in collapsed facades in Christchurch following the 2010/2011 earthquakes [4], where long embedment screws or throughout anchors were used.

CONCLUSIONS

Shake-table tests were undertaken in order to experimentally validate simple and cost-effective seismic retrofit solutions for solid clay-brick URM walls and the following conclusions were drawn:

- The critical failure mode for URM walls in the as-built condition was one-way bending in the out-of-plane direction with crack formation at three quarter-height enabling the wall to act as two separate rocking bodies.
- All of the tested retrofit solutions increased the PGA resisted by the wall as well as reducing the lateral displacements experienced up the height of the wall. The most effective mitigation system was the use of 90 x 45 mm timber strong-backs from wall base to parapet top, which

allowed flexural behaviour with a significant reduction in displacement and an increased PGA of three times the as-built condition.

- The use of 90 x 90 mm timber strong-backs further decreased the lateral displacement experienced, showing rigid-body behaviour. The parapet failure induced earlier instability with respect to the dynamic loading sustained by 45SB-p.
- Timber strong-backs were the most cost-effective and simple to install securing technique implemented. Standard 90 x 45 mm timber framing can be used as strong-backs, and do not require a specialist construction contractor to install.
- The roof diaphragm interaction with the wall provided a weak plane for cracking to form and the parapet to fail.
- Mechanical screw ties provided adequate wall-to-roof diaphragm connection during dynamic loading. Brick pull-out was observed prior to screw pull-out from bricks.
- Prestress losses associated with URM wall retrofit using threaded steel rods would eventually cause the wall to behave similarly to the as-built condition, resulting in one-way bending failure. Therefore, prestressed steel rods must be periodically checked and restressed if required. It was also observed that with reversing cycles of shaking the weak lime based mortar joints settled/compacted significantly, reducing the effectiveness of the vertical steel rods.
- External post-tensioned steel rods reduced installation cost and complexity and provided a good alternative securing technique, providing better results when compared to the mid-cross-section alternative, mainly due to significantly increased leaver arm as a result of external positioning.

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