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Dynamic behaviour of seismically retrofitted clay brick masonry cavity walls

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ABSTRACT: Loadbearing cavity (aka hollow) wall construction is a form of masonry wall construction where two leaves of unreinforced clay brick masonry (URM) are separated by a continuous air cavity and are interconnected using some form of tie system. An informative background to URM cavity walls is presented herein, including historical development, typical construction details, and observed earthquake damage. Shake table experimental testing of five walls that closely mimicked in-situ conditions for the most commonly encountered URM cavity wall arrangements was undertaken and the test methodology and results are presented herein. Mortar strength, retrofit type screws and spacing were the parameters investigated. Cavity walls were tested both in as-built and retrofit condition, considering the application of screw ties in the wall cross section and the combination of both timber strong backs and screw ties as retrofit intervention. During the test, cracking appeared mainly in the top quarter of the wall height, and differential movement of the two wall layers was observed only in as-built, showing the low strength of original ties. The use of screw ties lead to an improvement in seismic capacity of 30% to 60% depending on retrofit tie spacing. The performance of retrofitted walls further increased with the addition of timber strong backs, attaining a PGA two times higher the walls retrofitted using screw ties only.

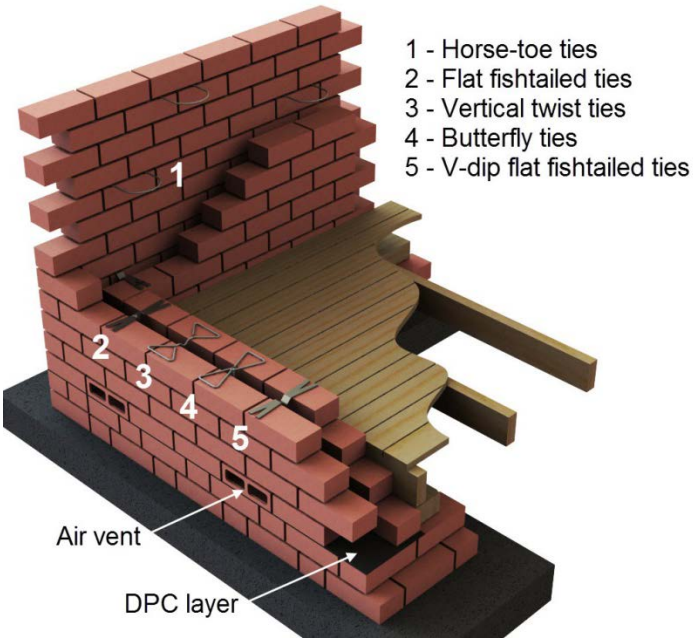
INTRODUCTION

The term 'hollow wall' was firstly defined in Nicholson (1852) as "*a wall built in two thicknesses, having a cavity between, either for the purpose of saving materials, or to preserve a uniform temperature in the apartments*". According to Hamilton (1959) in United Kingdom early hollow wall were 230 mm thick with 10 mm or 76 mm cavity where the two masonry leaves were interconnected using solid clay brick headers. The common size of clay bricks used had typical dimensions of approximately 230 x 110 x 75 mm. In order to address the problem with moisture penetrating via the header bricks, bended (Dearn 1821) or extruded (Gwilt 1888) header bricks were initially used. As reported in (Dizhur et al. 2015) such extruded brick detailing have not yet observed in New Zealand cavity walls, where this construction technique was used primarily between 1880 and 1935 (Russell and Ingham 2010).

The use of header bricks as inter-wall connectors was rapidly superseded in the 1840s by the use of metal ties, with early metal ties typically made of cast-iron or wrought iron, either untreated or dipped in tar and sand before being laid in the wall (see **Figure 1a**). Durability became a common issue with metal cavity wall ties because of their susceptibility to corrosion (see **Figure 1b**), hence builders began to use ties having copper welded to their surface or made of hot dipped galvanized steel (McKenzie 2001). **Figure 1c** shows the various shapes of the galvanized ties. Post-earthquake observations of damaged Christchurch buildings having URM cavity wall type construction, and where tie typologies were visible, showed that horse-toe metal ties were the most common type with a typical tie spacing being approximately 690 mm horizontally and 450 mm vertically (Dizhur et al. 2015).

(a) Typical cavity wall cross-section showing metal ties

(b) Corrosion of cast iron tie



(c) Different configurations of galvanized cavity wall ties

Figure 1. Examples of cavity wall ties in different configurations

In New Zealand, cavity construction was believed to be less common when compared to solid wall construction. Post 2010-2011 Canterbury earthquakes observations indicated that cavity construction was present in a large proportion of Christchurch URM buildings (Dizhur et al. 2015). **Table 1** is based on surveys conducted in Christchurch, Auckland (Walsh et al. 2014) and a preliminary survey of limited extent undertaken in the Dunedin area.

Table 1. Cavity URM construction for major cities as a percent of all URM buildings (Dizhur et al. 2015)

Survey region	Cavity URM construction
Auckland	40% (900) *
Christchurch CBD	20% (370) **
Christchurch outside CBD	50% (180) **
Dunedin	40% (50) ***

* Based on study by (Walsh et al. 2014)

** Most buildings were demolished following the 2010/2011 earthquake sequence

*** Based on relatively small sample size of URM buildings

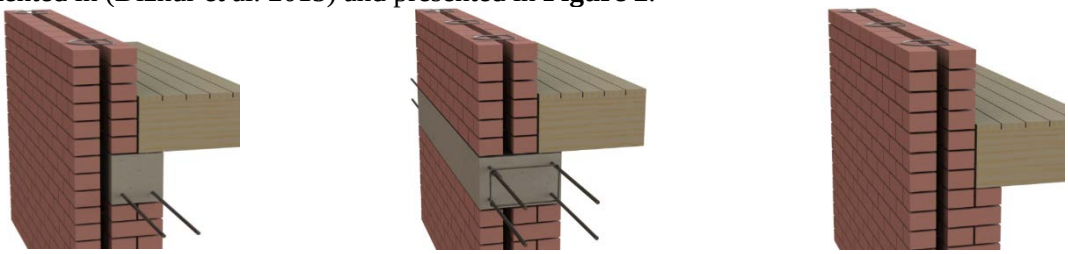
Note: Data rounded to nearest 10. Data in parenthesis indicates sample size

1 CONSTRUCTION DETAILS

In most buildings two single masonry leafs of brickwork would be separated by only a 50mm cavity (see **Figure 1b**). Nevertheless it was recognised that the thicker the masonry and the wider the cavity between the leaves (till 100 mm), the more effective the protection is against moisture penetration to the internal space, resulting in a warmer enclosed air space. Another commonly observed detail was to have a double loadbearing inner leaf, while less frequent was the use of a double-double leaf wall separated with an air cavity, (Dizhur et al. 2015). Some cavity walls were built with a damp proof course (DPC) leaf (see **Figure 1b**) above ground level to prevent moisture transfer from the foundation to the wall above. Often weep-holes are present to allow water that penetrated the outer leaf to drain outwards (Adams 1907).

Gravity load from the upper floors and roof was typically supported by the inner wall leaf of the wall as this mechanism enabled floor diaphragms to be kept clear of the damp external leaf, although in

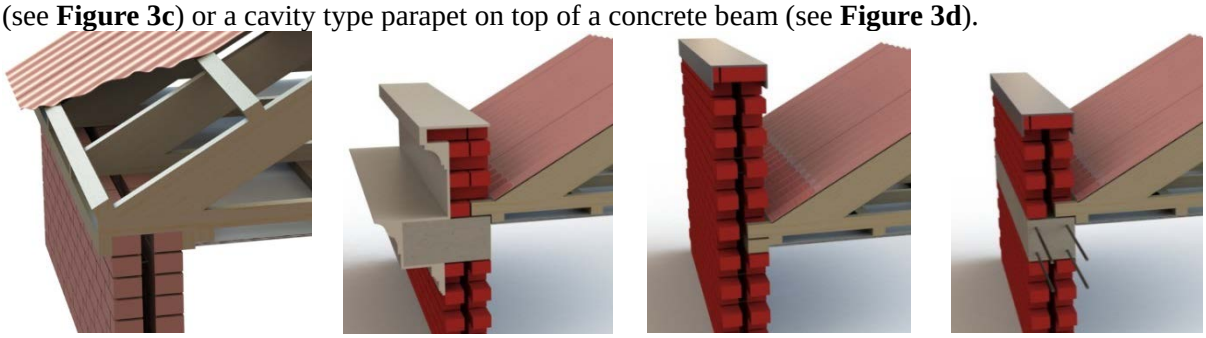
brick-tied construction the vertical gravity load is more likely to be shared between both wall leaves (Pickles, Brocklebank, and Wood 2010). The response of a cavity wall structure when subjected to lateral loads is dependent on the effectiveness of various diaphragm-to-wall seating arrangements. The three most prevalent types of floor diaphragm-to-wall seating arrangements for cavity walls are documented in (Dizhur et al. 2015) and presented in **Figure 2**.



(a) RC beam on inner wall leaf (Type-1) (b) RC beam extending across both inner and outer wall layer (Type-2) (c) Continuous outer and inner wall leaf (Type-3)

Figure 2. Typical arrangement of cavity wall at the floor diaphragm-to-wall connection

The most commonly encountered roof seating arrangements configuration is shown in **Figure 3a**. This connection type often provides some degree of restraints to the top of the wall that reduces the risk of out-of-plane cantilever collapse. Other cases involve the presence of unreinforced masonry parapets, non-structural and decorative element that extends above the roof around the perimeter of a building. **Figure 3b** shows a solid wall type parapet over a cavity wall where the roof framing seating arrangement is located at the transition from cavity to solid construction. In the case of cavity type parapet, the cavity construction could be full height with the roof framing seating in the interior leaf (see **Figure 3c**) or a cavity type parapet on top of a concrete beam (see **Figure 3d**).



(a) Typical roof diaphragm-to-wall connection (b) Solid wall type parapet over cavity wall (c) Continuous cavity type parapet (d) Cavity parapets with a concrete beam at roof level

Figure 3. Typical roof diaphragm-to-wall connection and parapet details

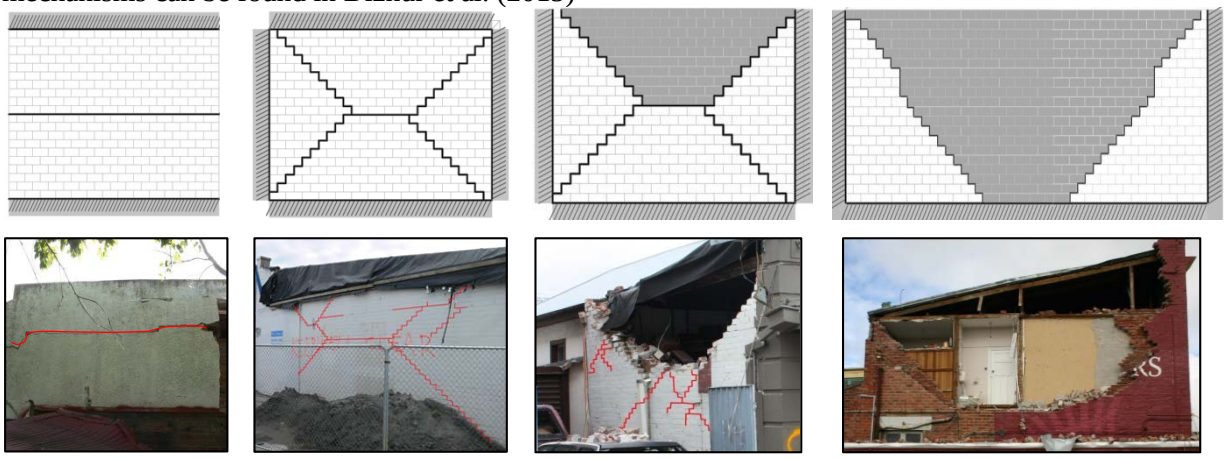
More details on the construction of cavity walls and site investigation including type of mortar used and cavity wall tie typologies, spacing and condition are presented in Dizhur et al. (2015).

2 SEISMIC PERFORMANCE

Following the 1989 Newcastle earthquake (Australia, M_L5.6), Griffith (1991) reported that out-of-plane failure of URM construction, typically cavity walls, was critical in terms of structural stability and safety issues and collapse of the outer masonry cavity wall layer due to wall bending failure was commonly encountered. It was also observed poor quality of masonry construction and mortar strength (Page 1996) and cavity wall ties often corroded, resulting in a poor connection between the two masonry wall layers. Therefore, the capacity of wall ties to transfer lateral load was significantly diminished due to cross-sectional reduction.

An inventory of frequently occurring failure mode types in cavity walls construction was undertaken by (Dizhur et al. 2015) considering 125 URM buildings damaged during the 2010/2011 Canterbury earthquakes (New Zealand). Out-of-plane failure causes most of the devastating structural damage in cavity wall buildings. Dizhur et al. (2015) classified the out-of-plane failure mechanisms into three types: (1) one-way bending (vertical bending of the wall), **Figure 4a**; (2) two-way bending, **Figure**

4b, and (3) cantilever response, **Figure 4c,d**. As shown in **Figure 4** the type of failure is strictly related to the end constraint condition, as a cavity wall with lateral end restraints possesses higher strength and stability than that with flexible end restraints. Boundary conditions steer also the failure of gable end walls or the out-of-plane failure of only the outer leaf. Examples of other observed failure mechanisms can be found in Dizhur et al. (2015)



(a) One-way bending (b) Two-way bending (c) Cantilever U-type failure for short spanning walls (d) Cantilever U-type failure for long spanning walls

Figure 4. Out-of-plane failure mechanisms observed on cavity wall construction

During the surveys it was identified that 20% (125) of 627 buildings surveyed following the 2010/2011 Canterbury earthquakes were of URM cavity type construction, most of them were located outside the Christchurch CBD where the proportion of cavity versus solid construction reached almost 50%. Considering only the cavity walls buildings, out-of-plane failure was registered in 72% of the buildings while in-plane failure in the remaining 28%. Within the out-of-plane failure, 7% was identified as one-way bending, 57% was recognised two-way bending and 36% had cantilever response (Dizhur et al. 2015).

3 SHAKING TABLE EXPERIMENTAL TESTING

Shake table experimental testing of five walls that closely mimicked in-situ conditions for the most commonly encountered URM cavity wall arrangements was undertaken. This experimental program added to the associated platform of knowledge reported in earlier sections herein and provided the research team with the needed data to validate/improve currently available assessment techniques for URM buildings with cavity type walls when loaded in their out-of-plane direction in either original as-built or seismically improved condition. Detailed description of the tested cavity walls, experimental results, observations and discussions are presented herein.

3.1 *Experimental programme*

All walls were constructed on a shake table platform and left to cure for at least 28 days prior to testing. The dynamic tests were performed on a dedicated shake table capable of applying sinusoidal horizontal loading. Wall W1 was tested in the as-built condition to serve as the control and allow the level of performance improvement in strength and displacement capacity to be quantified for retrofitted W2, W3 and W5 walls. Wall W4 was first tested in the as-built condition (W4.1) and then retrofitted with additional ties and retested (W4.2). Typically two tests were conducted for each wall (except wall W3 and W5). The first test (denoted as W#.1) was conducted where the table acceleration was increased incrementally up to a point of wall cracking and initiation of rocking and the test was stopped. For the second test (denoted as W#.2) the table acceleration was increased incrementally up to the point of wall collapse. **Table 2** summarises the characteristics of the tested walls.

Table 2. Cavity wall test matrix

Wall ID	Test ID	Height (mm)	Total wall Thickness (mm)	Mortar mix *	Retrofit type **	Retrofit tie spacing, V & H (mm) ***	
W1	W1.1 W1.2	3000	270	0:1:3	Original wire ties	V - 450	H - 600
W2	W2.1 W2.2	3000 2800	270	0:1:3	Φ12mm screws (Type 1)	V - 400	H - 600
W3	W3.1	3000	270	0:1:3	Φ12mm screws (Type 1)	V - 300	H - 600
W4	W4.1 W4.2	3000	270	1:2:9	Original wire ties Φ 8mm screws (Type 2)	V - 450 V - 300	H - 600 H - 600
W5	W5.1	3000	270	0:1:3	Φ12mm screws (Type 1) and timber strong backs	V - 400	H - 600

* cement:lime:sand; ** Φ – screw diameter in mm; *** V – vertical spacing; H – horizontal spacing. Note: All test walls were 1190 mm wide.

3.2 *Test walls construction and retrofit solutions*

Test walls were constructed as single-single masonry layer arrangement in a running bond pattern with a mortar joint thickness of approximately 10-15 mm. Recycled vintage clay bricks obtained from a demolished vintage URM building constructed in the 1930's were used. Recognising that within a building there is significant variability in brick properties, the reuse of vintage bricks introduced realistic material variability into the tests. Brick dimensions were of standard size (230L × 110W × 75H mm) that is commonly encountered in NZ heritage construction (Russell and Ingham 2010). Two different mortar mixes were used for the construction of the test walls, being 0:1:3 (W1-W2-W3-W5) and 1:2:9 (W4) (cement:lime:sand) by volume. The first mortar mix (0.25 MPa) was selected to simulate severe weather deteriorated mortar in vintage URM buildings, while the second one to simulate vintage cavity walls with moderate mortar strength (3.2 MPa). Portland cement was becoming more widely available in the early part of the 20th Century when URM was the building material of choice in NZ, as was hydrated lime. Hence Standard Portland cement, hydrated lime (Calcium Hydroxide) and river sand were used in the mortar. The results of material tests including compression test of bricks and mortar samples, compression and flexural bond tests of masonry prisms are presented in **Table 3**.

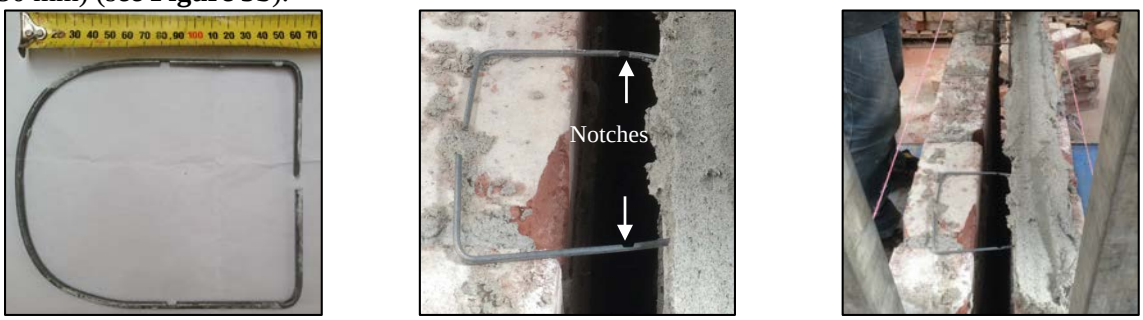
Table 3. Average material properties

Parameter	Symbol	0:1:3 mix Strength (MPa)	0:1:3 mix COV (%)	1:2:9 mix Strength (MPa)	1:2:9 mix COV (%)	Test method
Mortar compression	f'_j	0.5 (7)	26	3.2 (5)	33	(ASTM C109 2008)
Brick compression	f'_b	26.4 (10)	22	26.4 (10)	22	(ASTM C1196 2014)
Masonry compression	f'_m	5.60	-	15.5	-	
Masonry bond	f'_{fb}	0.08	-	0.10	-	Derived using (Almesfer et al. 2014)
Cohesion	c	0.1	-	0.5	-	
Coefficient of friction	μ	0.65	-	0.65	-	

(#) – number of samples tested

Wire cavity ties interconnecting the masonry layers were used in all the walls in order to simulate the 'as-built' condition that would be typically encountered in an actual vintage URM cavity wall building. Cavity wall ties were folded into the horse-toe wire tie geometry with notches cut into the wire (see **Figure 5a**) to simulate the rusted deteriorated condition of typical ties, as previously reported in §Introduction, and were laid based on the most common tie arrangement that was observed

in surveys, being two ties per row (600 mm) and one row of ties for every six masonry courses (450 mm) (see **Figure 5b**).



(a) Notched 'original' wall wire ties (W1) (b) Tie placement (W1)

Figure 5. Replicated wire ties using notched 4 mm diameter steel wire installed in all test walls

Based on previous airbag testing of cavity walls (Walsh et al. 2015), Φ12 mm mechanical screw ties were identified as the most effective retrofit solution in terms of increasing the out-of-plane capacity of cavity walls when comparing with stainless steel helical rods and chemical ties. The mechanical screw is a product that was originally commercially developed to be used for anchoring into concrete substrate and consists of a 12 mm diameter steel threaded shank with a length of 230 mm, **Figure 6a**. The screws had a total threaded length of 160 mm and a hexagonal washer type head. These screws were installed in walls W2, W3 and W5 using a pre-drilled 12 mm diameter hole in both wall layers. When installed, the tip of the screws reached at least half depth of the bricks on the opposing masonry layer. The installation process and final appearance is shown in **Figure 6b-e**.

A second type of mechanical screw tie was specifically developed to be used as a cavity tie in clay brick masonry walls. The mechanical screw consisted of an 8 mm diameter metal threaded shank with a length of 240 mm, **Figure 6a**. The screw has a fully threaded length and a Torx type screw head which allows the head of the fastener to be relatively small for the required torque. The diameter of the screw was selected in order to provide sufficient shear transferring capacity between the two wall layers and the thread pattern was adopted in order to optimise direct pull-out capacity and reduce the required installation torque. In wall W4 these screws were installed into a pre-drilled 7 mm diameter hole in both wall layers, **Figure 6e**.

Two screws were installed per row and each screw was 300 mm from the nearest edge, resulting in a 600 mm horizontal spacing. The vertical spacing arrangement was 400 mm in walls W2 and W5, and 300 mm in walls W3 and W4. Wall W5 was retrofitted using two strong backs utilising standard 90x45 mm timber planks at 550 mm spacing. Typically such timber framing is installed in order to provide support to the wall lining and are usually non-structural elements The timber planks were fixed to the wall using steel angle brackets applied in location of the Φ12 mm mechanical screw ties (400 mm spacing), see **Figure 6f**.

(a) Retrofit screws used, Type-1 above and Type-2 below (b) Hole drilling (c) Positioning of Φ12 mm steel screw (Type 1)

(d) Final installation of Φ12mm screws (Type 1)

(e) Finish of the screw, Type 1 above and Type 2 below

(f) Finish of the timber strong backs and Type 1 screws retrofit

Figure 6. Installation process of retrofit steel screw ties for all the walls, and timber strong backs for W5

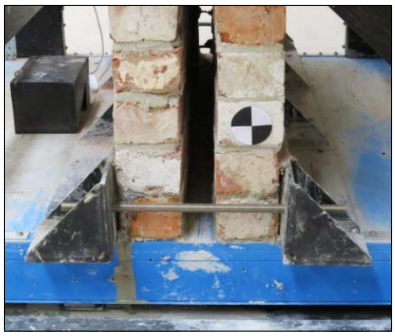
3.3 *Test setup*

For all the wall tests, the harmonic motion of the shake table was applied with increasing acceleration every 10 seconds. For each wall test the table acceleration was increased by increasing the speed of the electric motor by 0.5 Hz every 10 seconds, starting from 2.5 Hz. The amplitude of table motion remained constant at 50 mm. Two protection frames were designed and built onto the shake table platform, in order to provide wall top restraint and to protect testing instrumentation. The top wall restraint was a flexible hinge restraint that enabled the top of the wall to freely rotate and uplift after major cracks formed and rocking of the wall was initiated (see **Figure 7a**). This restraint was later adjusted for wall tests W2.2 and wall W3, W4, and W5 using horizontal beams applied to both sides of the wall and in the cavity in order to enable rotations and prevent horizontal translations of the top support (see **Figure 7b**). Two stiff steel angles were used to restrain the wall at the base, and grout was used to fill any gaps between the wall and the angles to ensure that lateral movement of the wall base was not possible (see **Figure 7c**).

Four accelerometers were installed at bottom, middle and top height of the wall and on the shaking table, and three draw wires were attached at middle and top height of the wall and on the shaking table to measure the differential displacement.

(a) Top wall restraint, W1-W2

(b) Top wall restraint, W3-W4-W5

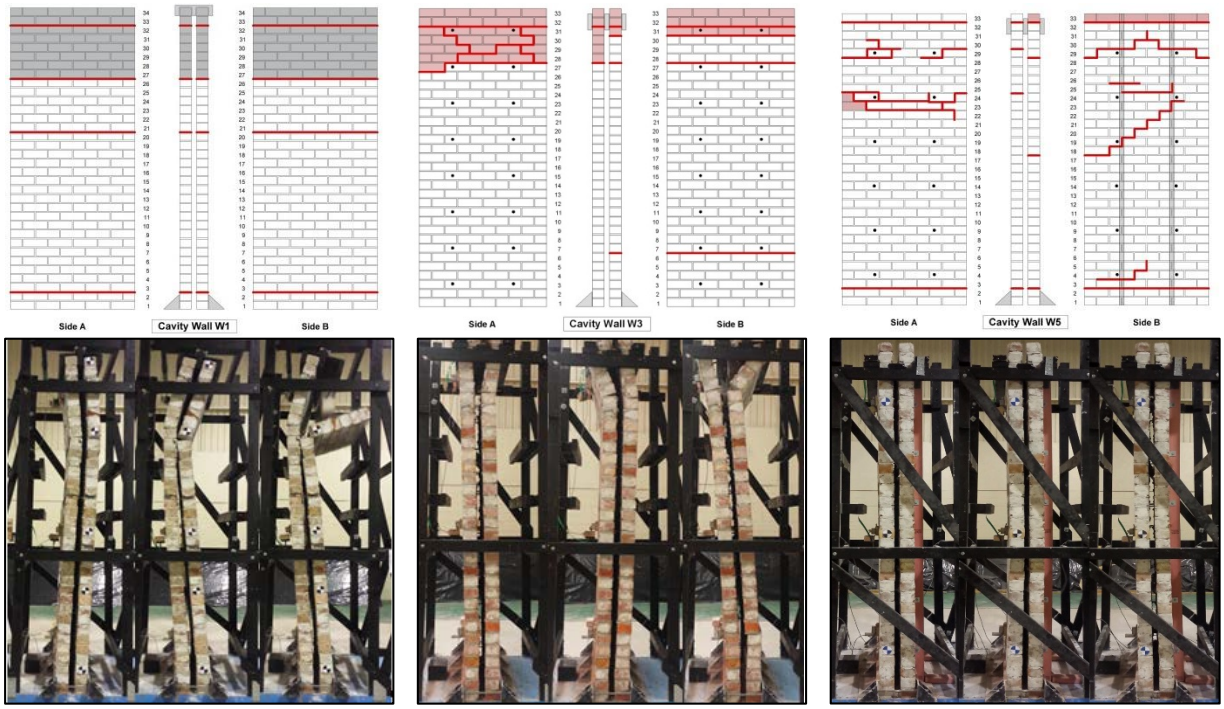


(c) Bottom wall restraint, all walls

Figure 7. Typical wall boundary restraints

3.4 *Experimental results*

Cracking was mainly concentrated in the top of the walls, involving approximately the top quarter of the wall height, see **Figure 8**. Differential movement of the two wall layers was observed in as-built wall W1. At the failure state it was observed that the width of the cavity between the two wall layers was significantly reduced, indicating the low compressive strength of the original cavity ties, see **Figure 8a**. In addition, the relative displacement capacity of wall W1 was limited due to weak mortar strength and major cracks fully opened through the wall following a few cycles of stable rocking during the test, resulting in collapse of the top.

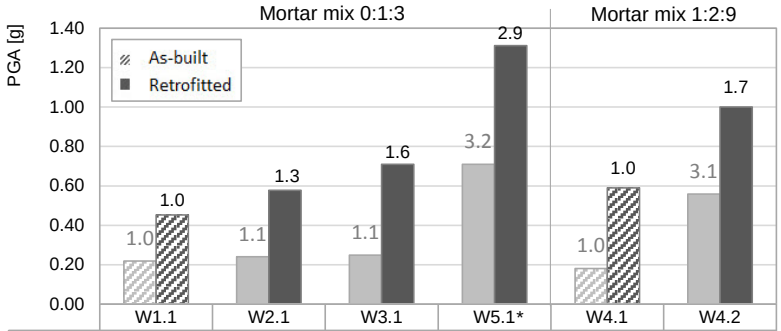


(a) W1 (b) W3 (c) W5

Figure 8. Screenshots showing crack-pattern survey and failure progression. Cracks and failure areas marked in red were detected during the first test, while cracks marked in black and grey areas were observed following the second test

The retrofitted walls W2, W3 and W4 failed in a similar manner as the as-built W1 wall, but responding as a rigid body. The crack-pattern was concentrated in the top part of the wall, involving the masonry above the highest one or two row of ties, **Figure 8b**. Hence the amount of masonry involved in the failure is related to the spacing of the ties, above 24 course in W2 and above 26 course in W3. Due to increasing bending and shear from uplift and rotation in conjunction with rocking of the wall, the top section failed because the mortar strength was exceeded. W4 was tested both in as-built and retrofit condition, showing a limited crack pattern due to the strong mortar used. In wall W5 the timber strong back allowed limited displacement at mid-height resulting in the initiation of a flexural behaviour at high level of ground acceleration (see **Figure 8c**). Instability or collapse of wall W5 was not attained even at the maximum possible load generated by the shaking table.

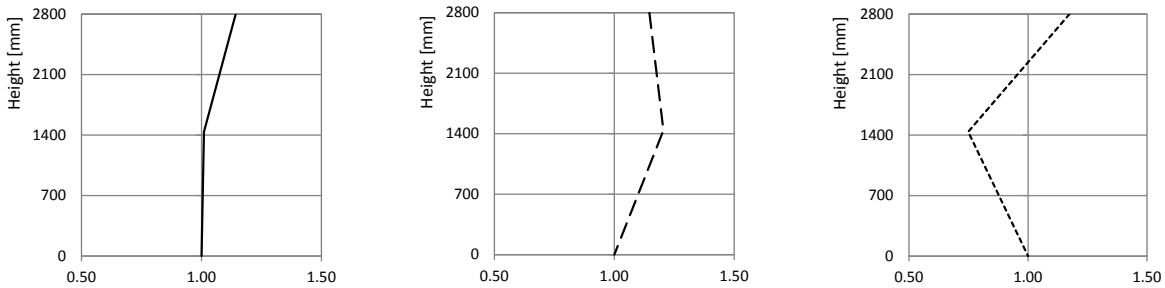
Figure 9 shows the PGA achieved by each wall at two different stages of wall behaviour, where the hatch pattern fill (W1.1 and W4.1) indicates the as-built walls. PGA achieved for initial cracking is shown as hatched in light grey and PGA corresponding to the initiation of rocking is shown as hatched in dark grey for each wall tested. As-built walls W1.1 and W4.1 differs by the type of mortar, W4.1 was constructed using stronger mortar that resulted in the wall reaching a 30% higher PGA (0.59g) than was achieved for wall W1.1 (0.45g). The peak capacity of the cavity walls increased when retrofit screw ties were installed (W2.1, W3.1, and W4.2), with the magnitude of increase being inversely related to the tie spacing (W3.1 and W4.2). Walls with weak mortar and Ø12 steel screw ties achieved PGA values of 0.58g for W2.1 and 0.71g for W3.1, with an increment of approximately 30% and 60% above the capacity of as-built wall W1.1. The application of timber strong backs fixed using Ø12 steel screw ties, wall W5.1, achieve of 1.31g corresponding to a three time increase in PGA compared to the as-built condition (W1.1) and two time of the wall that was retrofitted only with Ø12 steel screws (W3.1). Has to be noted that the initial wall cracking stage was observed in W5.1 at 0.71g, same PGA that caused the initiation of rocking in wall W3.1. Wall W4.2 was constructed with stronger mortar and Ø8 screws ties (Type 2) and reached a PGA of 1.00g, being 70% higher than the capacity of the as-built wall W4.1. Nevertheless, this PGA was exceed of 30% by wall W5.1 built with weaker mortar but retrofitted with screws and strong backs.



* Flexural behaviour observed

Figure 9. PGA achieved for initial cracking (hatched in light grey) and initiation of rocking (hatched in dark grey) for each wall tested (values at the top of each column indicate ratio of improvement relative to appropriate as-built test)

An in-depth analysis enabled the amplification of the peak acceleration vs wall height and the lateral displacement vs wall height to be identified, with the acceleration values normalized with respect to the table PGA in order to simplify the comparison between the walls. As a first step, these analyses were carried out at three stages of the test corresponding to the three main conditions of the test wall: (1) un-cracked, (2) cracked, and (3) rocking, see **Figure 10**. In **Figure 10a** it is shown that the acceleration profile was relatively constant in the un-cracked condition, and that as cracking occurred the acceleration profile becomes more triangular with the maximum acceleration developing at the wall mid-height, see **Figure 10b**. Following the initiation of rocking, the mid-height acceleration significantly decreased, resulting in smaller acceleration than that of the table, see **Figure 10c**.



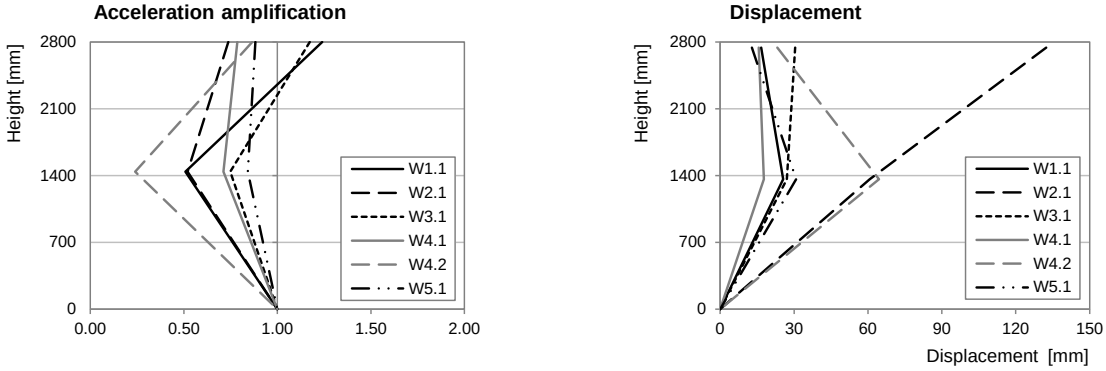
(a) Un-cracked behaviour (b) Cracked behaviour (c) Rocking behaviour

Figure 10. Typical acceleration amplification vs wall height at different stages for wall W3.1

A comparison between all the test walls considering only the acceleration profile at the final stage of the test (such as immediately prior to collapse) is shown in **Figure 11a**. All walls exhibited an acceleration decrease at the middle of the panel that ranged between 25% of the PGA for W4.2, 50% for W1.1 and W2.1, 70% for W3.1 and W4.1, and 85% for W5.1. At the top of the walls with weak mortar (W1.1, W3.1 and W5.1) the recorded acceleration was 120% of the table PGA and 85% in the case with strong back, and for walls with stronger mortar (W4.1 and W4.2) the recorded acceleration was 85% of the table PGA. The data recorded at the top of W2.1 was affected due to the damage pattern that resulted in cantilever type behaviour. This cantilever wall behaviour is clearly shown in the displacement graph in **Figure 11b**.

Figure 11b presents the relative lateral displacement of the wall vs height, indicating that the walls behaved as vertically oriented 'simply supported beams fixed at the base'. The maximum mid-height displacement that occurred in the walls with weak mortar (W1.1 and W3.1) was approximately 26-27 mm and the top displacement of W3.1 was approximately double that for W1.1 at 30 mm. W5.1 reached displacement comparable to W1.1 and W3.1 but at much higher acceleration, the displacement profile shows the flexural behaviour with an increment at mid-height of 18 mm when compared with the displacement in the top. Considering the test walls with stronger mortar (W4.1 and W4.2), rigid body behaviour was observed with the maximum displacement being similar at mid-height and top of the as-built wall, being 18 mm and 16 mm respectively. The addition of Type 2

screws ties resulted in accelerations that were higher than those developed by the as-built wall, with a maximum mid-height displacement of 65 mm and a 23 mm displacement at the top of the wall.

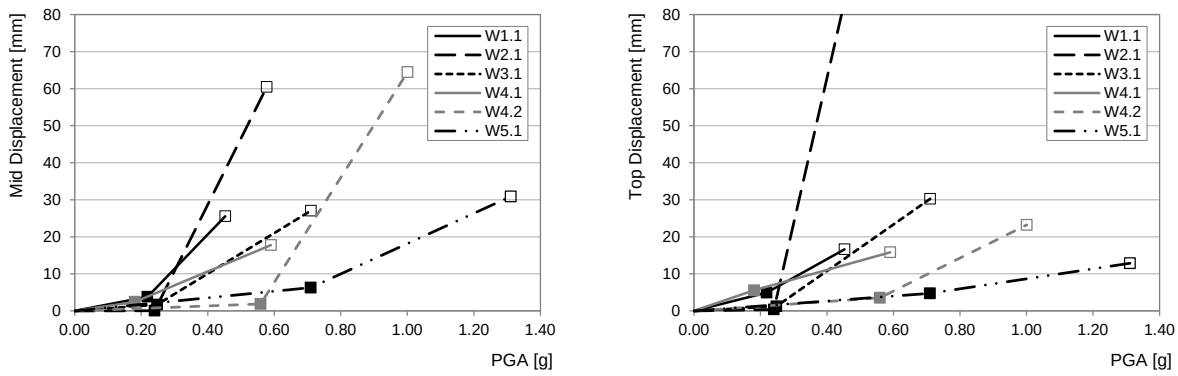


(a) Peak acceleration vs wall height normalized with respect to the table acceleration

(b) Displacement vs wall height

Figure 11. Peak acceleration and horizontal displacement vs wall height (data from W1.2 and W2.2 is not presented due to the malfunction of equipment)

As a final analysis, the trend of displacement versus the increase of PGA was evaluated both at mid-height and at top of the test walls. In **Figure 12** the PGA and displacement registered at initial cracking are indicated with black fill marks, while the rocking and consequent collapse at the maximum PGA is indicated by white fill marks. Large displacements were achieved at relatively small PGA in all the as-built walls. Both in W1.1 and W4.1 the initial cracking was observed at approximately 0.20g, corresponding to approximately 3 mm of displacement at the mid-height and 5 mm of displacement at the top of the walls. For retrofitted walls W2.1, W3.1 and W4.2, cracking initiated at 0.24g, 0.25g, and 0.56g respectively, with displacement at the mid-height ranging between 0.05 to 2.4 mm and between 0.4 to 5.5 mm of displacement at the top of the walls. For retrofitted wall W5.1, initiation of cracking was registered at 0.71g corresponding to a displacement of 6.3 mm at mid-height and 4.8 mm at the top of the wall. Based on the displacement at failure for as-built walls, the wall top-height displacement was approximately 17 mm in both the cases, while at mid-height were 26 mm and 18 mm respectively in W1.1 and W4.1, due to the different mortar strength. Retrofitted walls W3.1 presented similar displacement at the top and mid-height being approximately 29 mm, while W4.2 shows higher displacement at mid-height (65 mm) in comparison with 23 mm at the top. W2.1 results were altered due to the cantilever behaviour. Comparatively small displacements were exhibited at high PGA by W5.1, being 13 mm at the top and 30 mm at mid-height.



(a) Middle

(b) Top

Figure 12. PGA vs maximum displacement recorded in the walls. Black fill marks indicate PGA and displacement registered at initial cracking, while white fill marks show the rocking and consequent collapse at the maximum PGA

4 CONCLUSIONS

An informative background to URM cavity walls is presented in sequence of historical development, including construction details and seismic performance. In order to evaluate possible retrofit solutions

for cavity walls, shake table experimental testing of five walls that closely mimicked in-situ conditions for the most commonly encountered URM cavity wall arrangements was undertaken. Mortar strength, retrofit type screws and retrofit tie spacing were the parameters investigate, and led to the following observations from the shaking table testing.

- As-built walls (W1.1 and W4.1). Major cracks fully opened through the wall at 0.22g resulting in collapse of the top quarter of the wall at 0.45g (W.1.1) following a few cycles of stable rocking during the test. The use of stronger mortar (W4.1) led the reaching a 30% higher PGA (0.59g) in the as-built condition. The low compressive strength of original ties was clearly showed by the differential movement between the two wall layers. High displacements at relatively small PGA values were observed with 3mm to 26 mm of displacement at the mid-height and 5 mm to 17 mm at the top of the wall, respectively at initial cracking and initial rocking.
- Retrofitted walls using screw ties (W2.1, W3.1, and W4.2). A composite behaviour of the two wall layers, leading to rigid body behaviour was observed and cracks occurred above the highest one or two rows of screw ties involving several courses of bricks. The use of Ø12 steel screw ties, W2.1 and W3.1, improved the seismic capacity of 30% and 60% respectively comparing with as-built condition although the initial cracking occurred at similar PGA, 0.25g. The reduction of vertical retrofit tie spacing from 400 mm (W2.1) to 300 mm (W3.1) led to achieve a 20% higher PGA. The use of stronger mortar (W4.2) led the reaching of a 40% higher PGA than W3.1 in retrofit condition considering the same tie spacing but different screw ties type. The use Ø8 metal screw ties increase the seismic capacity of 70% comparing with as-built wall. In terms of displacement, the adopted retrofit system drastically reduced of five times at the top and four times at the middle-height the registered displacement at initial cracking, at equal PGA. In the case of W4.2, already damaged during as-built test W4.1, the displacement decrease of approximately 30-40%. At initiation of rocking, similar displacements were reached at much higher PGA.
- Retrofitted walls using screw ties and timber strong-back (W5.1). The addition of timber strong backs allowed the initiation of a flexural behaviour without attain instability. The seismic capacity increased of three times the as-built condition (W1.1) and two times the wall retrofitted only with Ø12 steel screws (W3.1). W5.1 shown the initiation of cracking at 0.71g, equal to the PGA reached from W3.1 at rocking but showing five times smaller displacements. Cracks were widespread trough the top half of the wall, although instability or collapse was not attained even at the maximum possible load generated by the shaking table.

High seismic performance were achieved by the proposed retrofit methods, in particular when the timber strong-back was added. Spread crack-pattern in the top-height of the walls allowed to dissipate the seismic energy inducing collapse at very high PGA. As expected, the seismic capacity increased reducing the spacing between the ties and increasing the mortar strength.

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