URM cavity walls:
Simple and experimentally proof-tested retrofit

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Abstract

Unreinforced clay brick masonry (URM) cavity-wall construction is common in seismically active parts of the world. This construction type incorporates a continuous air gap that separates the inner and outer brick leaves of the wall cross-section. Many past earthquakes (including the 1989 Newcastle, Australia earthquake) have highlighted that cavity-wall URM buildings perform poorly particularly in the out-of-plane loading direction due to their slender aspect and inadequate connections between masonry layers. An experimental campaign was initiated to develop a simple solution to substantially increase the out-of-plane performance of such walls by inducing composite action between the wall layers. Full scale URM cavity walls with varying cross-sections and retrofit arrangements were tested using a system of airbags to simulate earthquake loading. Wall construction details, retrofit procedures, test set-up, comparison with solid walls, and quantification of the improvement in seismic capacity from using the proposed retrofit techniques are presented herein.

Keywords: earthquake loading, airbag loading, unreinforced masonry, URM, seismic retrofit, cavity wall, cavity tie, clay brick masonry

1   INTRODUCTION

The critical engineering challenge when dealing with unreinforced clay brick masonry (URM) cavity walls is their stability when loaded out-of-plane. Each leaf of the wall is usually slender and is unable to sustain mid-height displacements beyond geometric instability of the single leaf without collapsing. It is now commonplace in current engineering practice to retrofit such walls with stiff

(shear transferring) steel ties which span between the leaves of the wall, crossing the internal cavity. A number of investigations have sought to verify and measure the performance improvement delivered by cavity ties. The focus of this research was on establishing an equivalent thickness of cavity walls by calculating the thickness of a solid wall which would perform nominally similar to a given cavity wall. Walsh et al (2015a, 2015b) undertaken in-situ testing of cavity walls found in existing buildings. It was concluded that 'cavity tie retrofits… can substantially improve the out-of-plane capacity of URM walls'. Following this work, provisional equations to quantify the equivalent solid thickness of cavity walls were proposed. Giaretton et al (2016a, 2016b) undertaken dynamic testing of cavity walls using a shake table. The testing showed that the slender as-built wire ties common to most cavity walls added no additional deflection capacity to the walls, resulting in the walls performing similarly to independent single leaf walls. Cavity walls retrofitted with shear transferring ties demonstrated 'composite rigid-body behaviour', supporting the theory that a rocking model could be applied to tied cavity walls.

Although researchers have observed that cavity wall typologies extend beyond walls of two single leaves (Dizhur, (2015); Giaretton, (2016a)), to the authors' knowledge there were no experimental testing of asymmetrical leaf arrangements in URM cavity walls. The research programme presented herein includes the testing of walls with a solid double leaf URM wall connected to a single leaf. This arrangement is common on lower floors of URM buildings and can also be found on upper floors.

2   WALL TEST SPECIMENS

The experimental testing programme was conducted using four purpose-built full-scale URM walls (see Figure 1a). The walls were 3000 mm high and 1150 mm wide, with wall height chosen to approximate a typical upper storey in a URM building. Two of the walls were single-single cavity walls, meaning two single leaves of bricks separated by a 55 mm wide air cavity. The other two walls were double-single cavity walls, meaning a double leaf of brick, a 55 mm air cavity, and a single leaf of brick. These walls were denoted "1+1" and "2+1" respectively. These type of wall configurations are frequently found in New Zealand URM buildings, with the double-single cavity walls commonly but not exclusively found on lower floors. The cavity width in all the walls was specified as 50 mm, or approximately half the thickness of a brick. As was common practice in historical URM construction, the outer faces of the walls were made flush. As a result, there was minor variability in the cavity thickness over the height of the wall.

In typical URM cavity wall construction, flexible wire ties were inserted into the walls to connect the leaves across the cavities. Following the methodology employed by Giaretton (2016c), these as-built wire ties were simulated in the specimen walls using 4.0 mm steel wire bent into open hoops. Deterioration of the as-built ties was simulated by notching the wire hoops which were built into the specimen walls (see Figure 1b). Wire hoops were inserted every six courses, alternating between a pair of hoops and a single hoop to create an offset pattern. In retrofitted wall specimens, Python C ties were used to transfer shear and improve composite action between the leaves. The retrofit ties were 6 mm diameter screws with a length of approximately 240 mm and have significantly higher axial and flexural capacity than as-built ties. Python C tie spacing was varied for each retrofit test.

The bricks from which the wall specimens were constructed were sourced from the demolition of a vintage URM building. The bricks are approximately 100 years old, and made from reddish-orange clay, typical of New Zealand URM buildings. Material testing was undertaken on single bricks and three brick high masonry prisms. The mortar used to construct the walls was mixed to a cement:lime:sand ratio of 1:2:9 by volume. This ratio was chosen to follow a mortar mix used in

previous cavity wall testing by Giaretton (2016c). Material testing followed ASTM standards (ASTM, 2013, ASTM, 2014a, 2014b).


(a) Wall specimens following construction (showing different wall typologies)
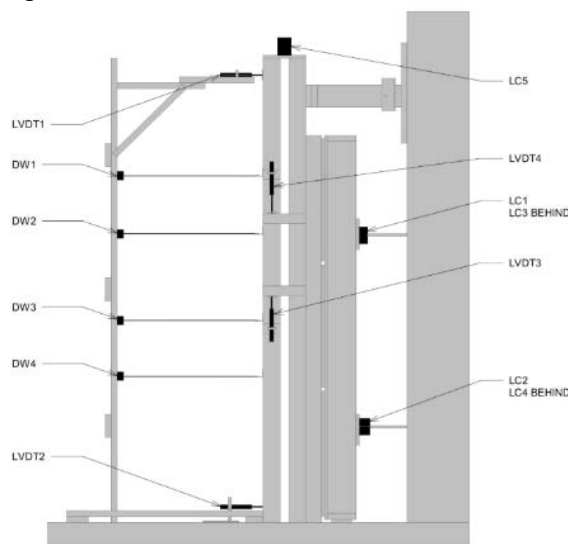

(b) Notched wire hoop simulating historical as-built ties.

Figure 1: Wall specimens and material testing

The mean compressive strength of the bricks in uniaxial compression was 22.7 MPa (14 samples, CoV 45%). This brick strength is between the benchmark value for the categories Soft and Medium, as defined in Table C8.3 of the New Zealand Guidelines for the Seismic Assessment of Existing Buildings (hereafter "the Guidelines") (MBIE, 2017). The mean compressive strength of the mortar cubes was 2.0 MPa (16 samples, CoV 45%). This corresponds to the upper end of the category Soft, as defined in Table C8.4 of the Guidelines. (MBIE, 2017). The mean compressive strength of the masonry prisms was 11.5 MPa (10 samples, CoV 19%). This compressive strength corresponds to a mid-range masonry compressive strength as defined in Table C8.5 of the Guidelines.
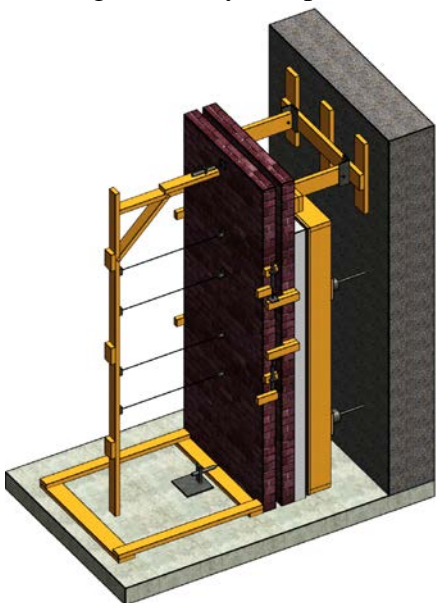
(a) Isometric schematic of setup   *(b) Side elevation showing instrumentation plan*   *(c) Wall under test*

Figure 2: Test setup and instrumentation plan

3   TESTING PROGRAMME

The wall specimens were tested using a system of airbags to apply a uniformly distributed semi-cyclic loading, with the test setup is shown in Figure 2a. Each wall was restrained at its base with timber

members bolted to a concrete strong floor and bearing directly on the first course of bricks. At the top of the wall specimen, a timber stringer was connected to brick course 31 using Python MT screws, and timber joists connected the stringer to a concrete strong wall. A loose bolted connection between the joists and the strong wall allowed free rotation of the top of the wall but prevented horizontal translation. The upper and lower connections were designed to recreate in-service conditions of URM walls in a post-retrofitted state and allowed each wall to be considered pinned at top and base.

The airbags reacted against the timber reaction frame seen in Figure 2 to the right of the URM wall specimen and were sandwiched between the timber and a layer of polystyrene (shown in white in Figure 2c) to ensure a better distribution of applied pressure. When airbags were inflated, a force was exerted on the wall specimen which was measured by four load cells, located on the opposite side of the timber reaction frame. Loading was applied in a gradual manner by controlling the airflow into the airbags, and multiple loading/unloading cycles were applied to the wall specimens. The standard instrumentation plan is shown in Figure 2b.

Table 1: Summary of testing programme and results

Wall (type[1])	Test # (retrofit[2])	Tie spacing[3] (mm)	Max displace-ment[4] (mm)	Max force[5] (kN)	Notes
W1 (1+1)	1 (as-built)	-	76	3.9	
	2 (10 C)	690 H / 700 V (8 courses) & 690 H / 500 V (5 courses) above course 17	112	5.8	
	3 (12 C)	690 H / various V (see Section 3.1)	118	5.0	
W2 (1+1)	4 (21 C)	460 H / 400 V (4 courses) & 460 H / 600 V (6 courses) above course 19	84	4.4	Displacement sequence halted to prevent break-off collapse (see Section 3.2)
	5 (23 C)	As Test 4 plus 2 ties course 22	104	2.7	
	6 (23 C)	As Test 5	149	3.4	
	7 (as-built)	-	54	1.1	
	8 (as-built)	-	90	-	Collapse test[6]
W3 (2+1)	9 (as-built)	-	53	8.4	Oriented 2+1[7]
	10 (21 C)	460 H / 400 V (4 courses)	153	6.9	Oriented 2+1[7]
	11 (24 C)	As Test 10 plus 3 ties at top support	193	8.6	Oriented 1+2[7]
	12 (as-built)	-	52	5.1	Oriented 1+2[7]
	13 (24 C)	As Test 11	230	-	Collapse test[6]. Oriented 1+2[7]
W4 (2+1)	14 (24 C)	460 H / 400 V (4 courses) plus 3 ties at top support	181	12.0	Oriented 1+2[7]
	15 (as-built)	-	109	7.4	Oriented 1+2[7]
	16 (as-built)	-	74	6.5	Oriented 2+1[7]
	17 (24 C)	As Test 14	103	9.2	Oriented 2+1[7]
	18 (24 C)	As Test 14	350	-	Collapse test[6]. Oriented 2+1[7]

1. Wall type 1+1 denotes a single-single cavity wall, 2+1 a double-single cavity wall. The 2+1 walls were tested in both orientations.
2. The figure in parentheses denotes the number and type of ties: eg "10 C" means ten Python C ties were inserted into the wall. "As-built" means no retrofit.
3. "H" denotes horizontal spacing; "V" denotes vertical spacing. Vertical spacing is also given in courses.
4. The maximum displacement is calculated using the draw-wire with the highest reading and extrapolating to the crack location. This is not the ultimate displacement that the walls can sustain but a displacement at which the test was terminated.
5. The maximum force is the sum of the forces measured by the four load cells
6. Note that to protect instrumentation, forces were not recorded for collapse testing
7. "2+1" denotes load applied to double leaf; "1+2" denotes load applied to single leaf

3.1 Wall 1 (single-single)

Wall 1 (W1) was a single-single (1+1) cavity wall with gross dimensions 3000 mm × 1150 mm × 275 mm. Wall 1 was tested in the as-built state through twelve cycles of increasing deflection, reaching a peak deflection of approximately 75 mm at the crack location between courses 19 and 20 (see Figure 3a). After each cycle the wall returned to its equilibrium position with minor residual displacement. Following cracking, the total peak load fluctuated at approximately 3.5 kN. W1 was then retrofitted using 10 proprietary shear transferring ties (Python C ties). The spacing of the ties was approximately 690 mm horizontal, 600 mm vertical. As before the wall was loaded in cycles with increasing deflections, reaching a peak displacement at the first crack initiation of approximately 112 mm. The total peak force increased from 3.9 kN in the as-built state to 5.8 kN following ties installation. Instruments recording the relative vertical displacement of the inner and outer leaves of the wall recorded an increase in differential movement when lateral displacement reached around 50 mm, indicating that the ties were undergoing some level of plastic deformation.
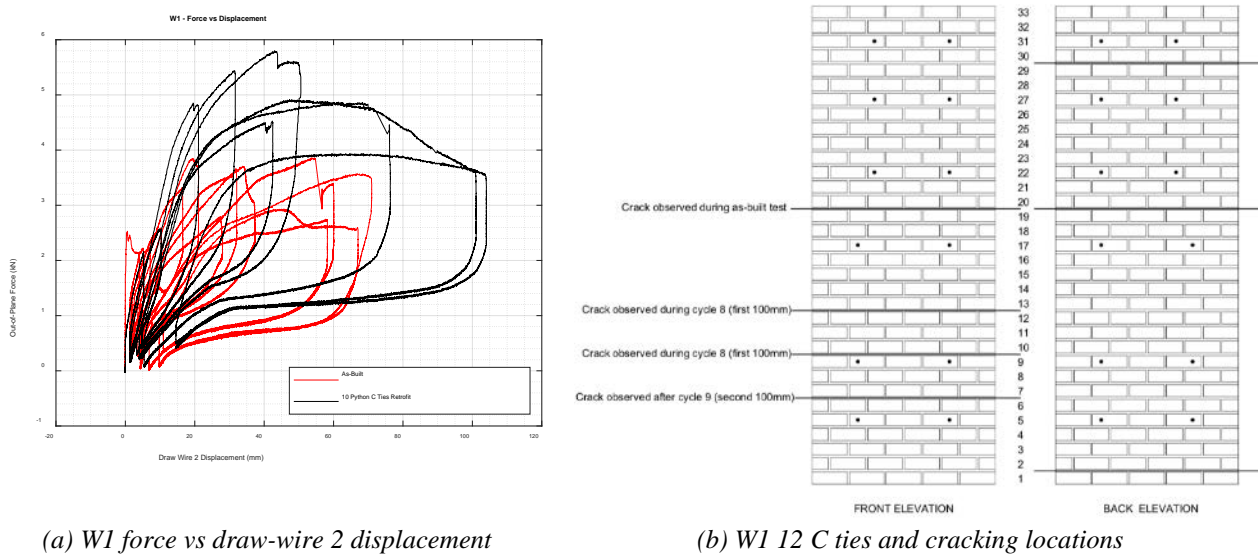
(a) W1 force vs draw-wire 2 displacement   *(b) W1 12 C ties and cracking locations*

Figure 3: W1 Load-displacement relationship, tie layout, cracking

For the final test of W1, two additional ties were added to the wall at course 5 to increase capacity and reduce demand on the overloaded ties at course 9. The wall withstood deflection to approximately 118 mm at the crack with a reduced peak force dropped to 5.0 kN (14% reduction). However, further cracking was observed in the lower part of the wall, with cracks opening up between courses 6 and 7 and between courses 12 and 13 (see Figure 3b). The distributed cracking of the wall into multiple blocks is a possible indication that the adopted tie spacing for W1 was too wide.

3.2 Wall 2 (single-single)

Wall 2 (W2) was a single-single (1+1) cavity wall of the same dimensions as W1. Because of the distributed cracking and tie flexure that occurred in W1, W2 was initially retrofitted with a more closely-spaced tie pattern in the lower half of the wall. Four rows of three ties were inserted below course 20, spaced at approximately 460 mm horizontally (every two bricks) and 400 mm vertically (every four courses). Above course 20 two rows of three ties were used, spaced vertically every six courses. During the 30 mm deflection cycle, the wall cracked in two places on the outer leaf, between two rows of ties. This cracking pattern indicates that bricks on the outer leaf between courses 19 and 25 were not connected to the rest of the cavity wall (see Figure 4a), threatening the break-off failure illustrated in Figure 8c.
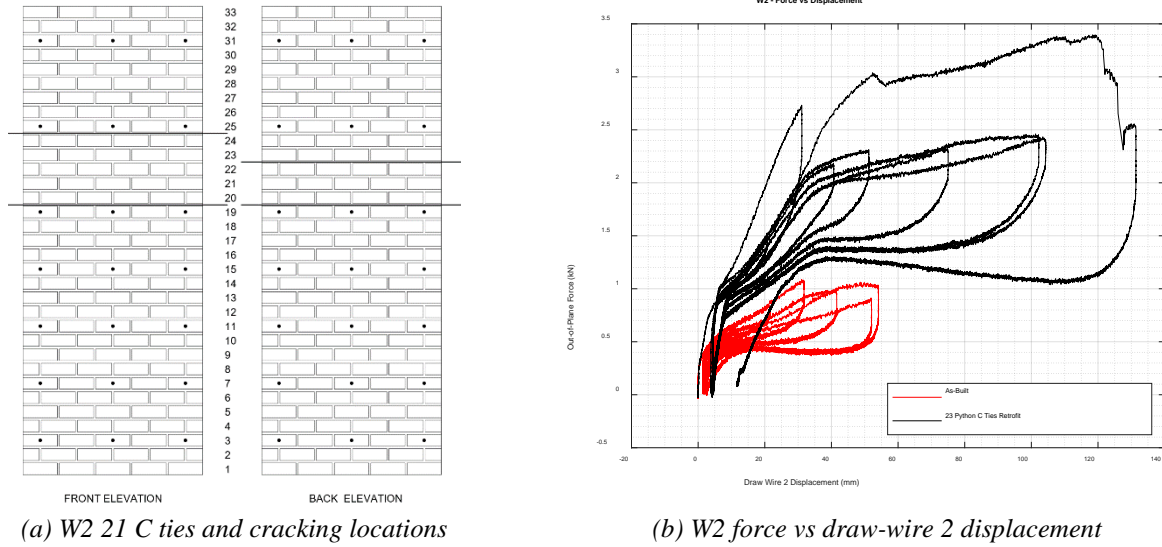
(a) W2 21 C ties and cracking locations   (b) W2 force vs draw-wire 2 displacement

Figure 4: W2 Load-displacement relationship, tie layout, cracking

Two additional ties were added at course 22 to secure the masonry upper block. With the addition of these ties the wall underwent rocking and recovery cycles up to displacements of approximately 100 mm (see Figure 4b). During the last cycle, the wall was displaced to 127 mm at draw wire 2, corresponding to a deflection of approximately 150 mm at the crack between courses 19 and 20. The wall then returned to a stable equilibrium position with minor residual displacement.

The cavity ties were removed from the wall and testing was resumed in an as-built condition. The total peak force applied during the as-built test was approximately 1.0 kN, with high levels of relative vertical displacement between leaves. During the final test, the wall was deliberately collapsed in order to establish a baseline for instability in as-built cavity walls. Collapse was sudden and occurred at a displacement of approximately 90 mm at the mid-height crack.
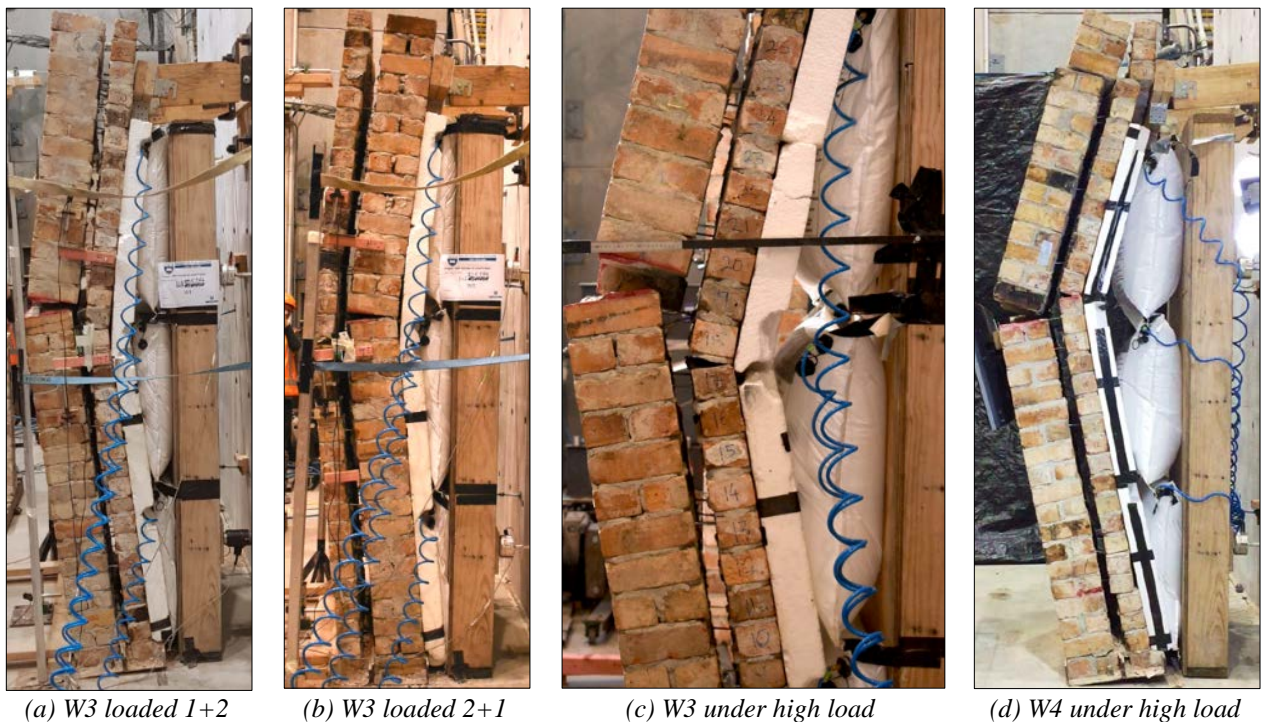

(a) W3 loaded 1+2   (b) W3 loaded 2+1   (c) W3 under high load   (d) W4 under high load

Figure 5: Double-single walls under loading

3.3 Wall 3 (double-single)

Wall 3 (W3) was a double-single (2+1) cavity wall with gross dimensions 3000 mm × 1150 mm × 395 mm. Double-single walls are necessarily asymmetrical, and it was predicted that their response would be anisotropic. As a result, the testing programme included turning each wall specimen so that it could be tested from both out-of-plane directions. Load applied to the double leaf orientation is herein denoted as "2+1". When loading is applied to the single leaf, the orientation is denoted "1+2" (see Figure 5a-b).

W3 was first tested as-built in the 2+1 orientation. Following initial cracking, the wall was cycled through increasing displacements up to approximately 50 mm, returning to near equilibrium position at the end of the as-built test. An LVDT measured high relative vertical displacement of the leaves of the upper block, with a peak displacement exceeding 20 mm (see Figure 6a). The wall specimen was retrofitted using 21 Python C ties at 460 mm horizontal centres and 400 mm vertical centres, corresponding to a row of ties every four courses. With ties in place, the crack from as-built testing between courses 17 and 18 did not initially reopen, but a new crack formed between courses 21 and 22 during testing. The wall reached approximately 155 mm at the mid-height crack and returned to an equilibrium position with minor residual displacement. During the 155 mm displacement cycle, the crack between courses 17 and 18 which had occurred in the as-built test reopened, and the block between course 18 and 21 moved semi-independently of the upper block, at a separate angle. The wall was able to return to a stable equilibrium, and it was subsequently able to sustain displacements in the opposite loading direction. Additionally, the upper LVDT measured a peak interleaf vertical displacement of approximately 4 mm (see Figure 6a) during the tie retrofit test, suggesting that the cavity wall leaves acted compositely as a semi-rigid body and relative displacement was limited.

The wall W3 was rotated 180 degrees and a new row of ties was inserted in course 31 to ensure a load path between the outer leaf and the upper restraint. The wall sustained high levels of displacement during the 1+2 retrofit test, peaking at approximately 165 mm for draw wire 2. Draw wire 3, connected at course 15 on the lower part of the wall, measured a peak displacement over 190 mm. For these large displacements to occur, the inner single leaf lifted and fully support the outer leaf via the retrofit ties (see Figure 5c). To establish a directional difference between performance of as-built double-single cavity walls, the wall specimen was tested again with ties removed, for comparison with the as-built test in the 2+1 orientation. The peak loading in both directions was similar at approximately 5.0 kN. Lastly, ties were reinstalled in wall 3 and it was loaded to collapse. The wall withstood displacement to 350 mm at the mid-height crack before collapsing, which represents a geometrically-determined instability failure rather than a tie failure.
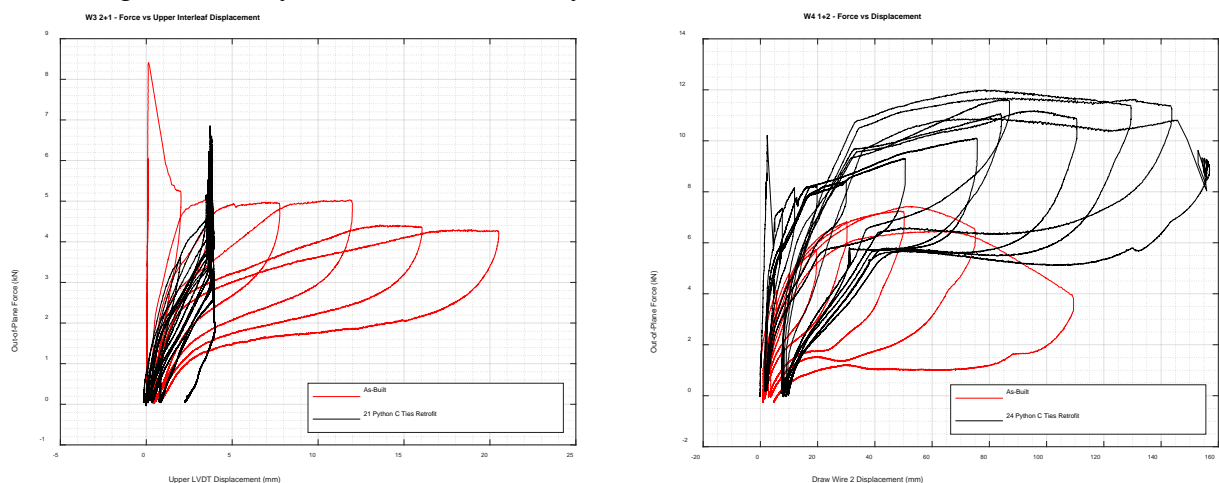
(a) W3 2+1 force vs relative vertical leaf displacement   *(b) W4 1+2 force vs draw wire 2 displacement*
Figure 6: Force vs displacement plots for double-single walls

3.4 Wall 4 (double-single)

Wall 4 (W4) had nominally same dimensions and testing setup as W3, except that testing began in the 1+2 orientation. The wall specimen was retrofitted with 24 Python C ties spaced at 460 mm centres horizontally and 400 mm (four courses) vertically. During testing in this orientation, the highest overall load capacity for the double-single walls was reached. Not only did the elastic pre-cracking force reach over 10.0 kN, but post-cracking loads consistently reached approximately 12.0 kN. These results correspond with theoretical predictions as the 1+2 orientation has the highest force capacity when analysed using the model in Section 4.2. In the displacement domain, the wall reached nearly 160 mm at draw wire 2 (see Figure 6b), corresponding to a displacement of approximately 180 mm at the crack between courses 18 and 19. The wall then returned to a near equilibrium position with minor residual displacement.

Ties were removed from the wall and displacement cycles commenced for testing in the as-built condition. Displacement of 110 mm was achieved with a rapidly-decreasing total lateral force, suggesting that instability would shortly follow. The wall was rotated to the 2+1 orientation and re-tested, without ties being reinstalled. Results for the as-built tests were similar with a peak load of 7.4 kN in the 1+2 orientation and 6.5 kN in the 2+1 orientation.

24 Python C ties were reinstalled in wall W4 and a series of displacement cycles was conducted. The maximum displacement reached was approximately 105 mm in the 2+1 retrofit condition. Peak loading reached a plateau of approximately 8.0 kN, an increase of 23% from the as-built load capacity of 6.5 kN in the previous test. However, this loading was consistently lower than that measured in the 1+2 test, suggesting that the shorter lever arms between the masses and the pivots affect lateral capacity as predicted by the model in Section 4.2. Finally, instrumentation was removed, and the wall was loaded in its retrofit condition until collapse at a displacement of approximately 350 mm, governed by reaching the geometric instability point of the wall (see Figure 5d, and Figure 7).

Figure 7: W4 collapse time sequence

4 RESULTS AND DISSCUSSION

The most effective tie spacing was found to be approximately 460 mm horizontal (ties every two bricks) and 400 mm vertical (ties every four courses). This spacing generally allowed walls to crack horizontally into an upper and a lower body. Each section maintained pseudo-rigid behaviour in which major cracking did not occur within each of the two blocks, tie deformation was minimal, and wall leaves rocked compositely. When break-off failure (see Figure 8c for a schematic) was encountered in W3, the ties allowed the wall to return to equilibrium and subsequently be cycled in

the opposite direction. Well-tied walls were able to sustain deflections to the point of geometric instability. Failures relating to tie modes were not observed in walls with ties spaced as above.

Substantial variation in the force-based capacity of as-built walls resulted from material variability, significant cracking damage, and restraint conditions. However, each wall in a tie retrofit condition had increased force capacity over the same wall specimen in its as-built state.

Displacement capacity of tied walls increased markedly over as-built walls. Semi-independent single leaves without shear-transferring ties, whether in 1+1 or 2+1 walls, reached instability at an upper limit of approximately 110 mm. Tied walls sustained large displacements, approaching 200 mm, without collapse.

Relative vertical displacements between inner and outer leaves were limited by cavity ties, demonstrating that it is valid to assume a load transfer path as described in Section 4.2 in which well-tied cracked wall bodies can be treated as having a single centre of mass close to their geometric centre.

Double-single walls were able to sustain greater lateral force in the 1+2 orientation than in 2+1 form. It is theorised that the larger distance between 1+2 walls' centre of mass and the point of loading creates greater restoring moment to resist lateral loads. However, the instability deflection limit in collapse tests was lower for the 1+2 orientation, perhaps because the central pivot must be supported on a single leaf.

By observation, the lower blocks of cracked walls did not exhibit classical rocking behaviour in the manner illustrated in Figure 8b. The wall bases tended to stay level, except under extreme lateral displacements. Cracking occurred a few courses above the base at moderate displacements, but classical rocking occurred at higher displacements and above the first few courses. The upper and lower wall sections demonstrated the self-centring behaviour encoded in the rocking model, and so the practical difference between pure rocking and the observed behaviour is small.

Based on the experiential results and subsequent analysis it is proposed that well-tied cavity walls can be treated as solid walls of the same gross outer-to-outer thickness for the purposes of practical engineering assessment. Cavity walls can be considered well-tied if the failure mode of the wall is not related to or caused by the ties. Failure modes of well-tied cavity walls include reaching instability deflection, bed joint shear, or failure of the wall-diaphragm connection. These modes represent the upper bound of well-tied cavity walls without additional strength from use of timber strong-backs, unbonded post-tensioning, or external frames, and they do not represent a limit upon the ability of the wall to rock. Failure modes of poorly tied cavity walls include excessive vertical deflection of the ties (leading to collapse of the outer leaf), shearing of ties, or break-off failure in which bricks below a row of ties fall away (see Figure 8c). Additional unacceptable failure modes relate to axial loading of the ties, including tie withdrawal or tie buckling. Experimental results show that an optimal tie spacing for well-tied walls is approximately 460 mm horizontal centres, and approximately 400 mm vertical centres (four courses).
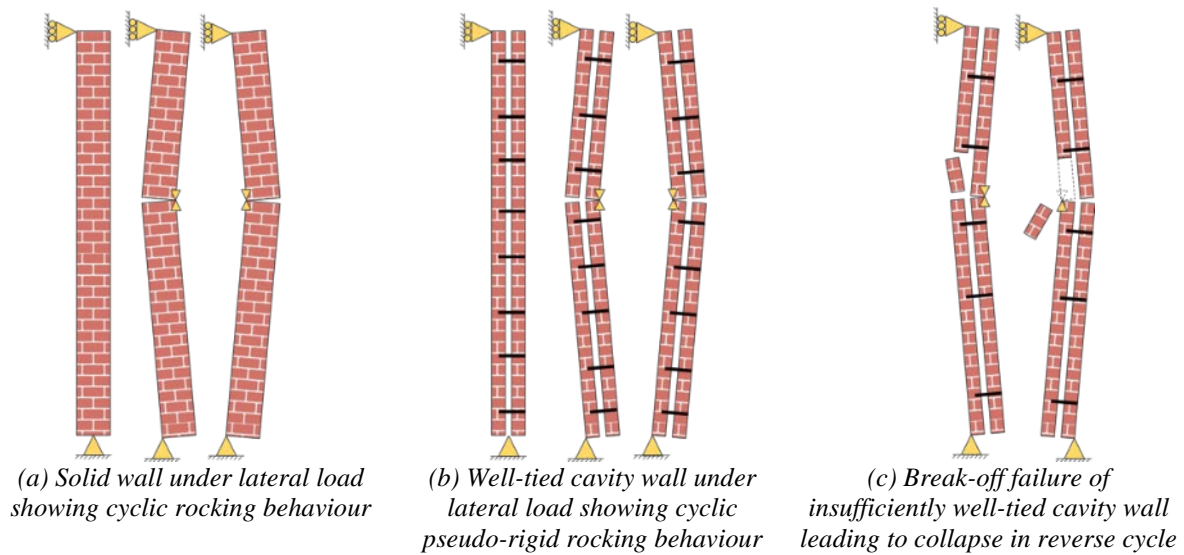
(a) Solid wall under lateral load showing cyclic rocking behaviour   *(b) Well-tied cavity wall under lateral load showing cyclic pseudo-rigid rocking behaviour*   *(c) Break-off failure of insufficiently well-tied cavity wall leading to collapse in reverse cycle*

Figure 8: Schematic drawings of rocking behaviour and failures

Walsh (2015a, 2015b) and others have provided calculations for determining the capacity of individual ties, on the basis of multiple potential tie failure modes, and have also offered equations to describe the effective thickness of cavity walls relative to solid ones. However, if walls are well-tied, tie failure modes are irrelevant in the assessment of wall capacity. Testing and calculations have demonstrated that axially-directed tie failure modes like pull-out and buckling are sufficiently far up the failure hierarchy that they are extremely unlikely to occur. For rocking to be sustained, flexure is the critical tie failure mode. The critical loading case for flexure is at the initiation of uplift, when the maximum proportion of weight force is directed perpendicular to the ties' central axis (see Figures 8b and 10a). The magnitude of this demand is related to the geometry of the wall, not to the out-of-plane load applied to the wall. Significant lateral loading will cause uplift of the outer leaf above the mid-height crack and thus will impose maximum demand upon the ties. Thus, the number of ties required for any predicted out-of-plane load is the number of ties required to allow rocking to occur through uplift of the outer leaf.
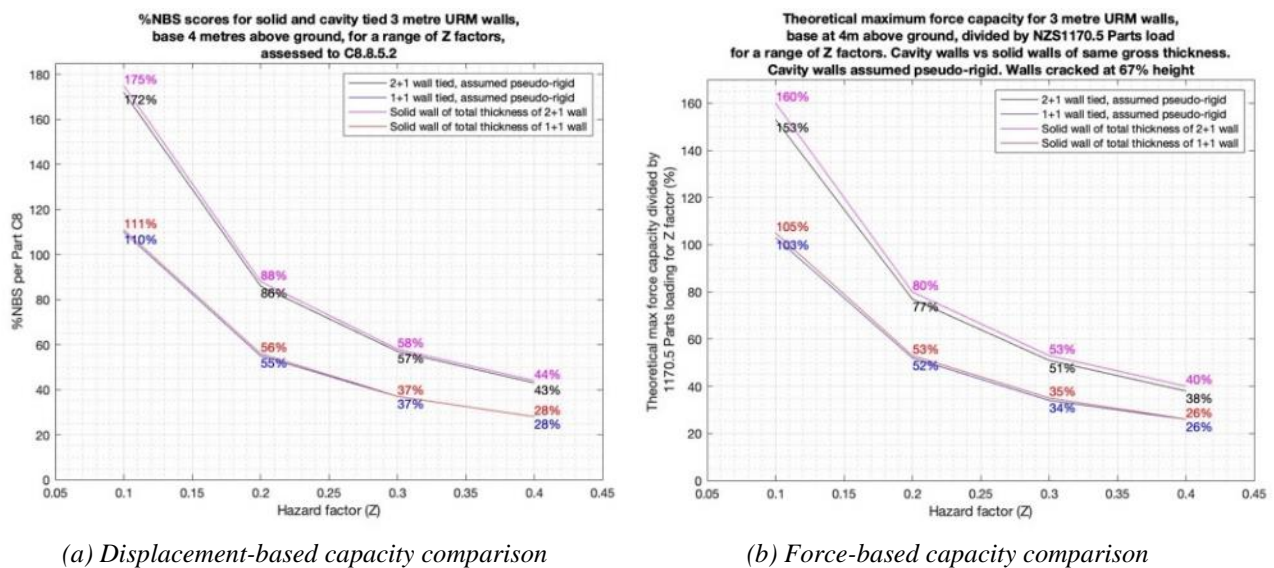

(a) Displacement-based capacity comparison   *(b) Force-based capacity comparison*

Figure 9: Comparison of calculated capacity of solid walls vs well-tied cavity walls

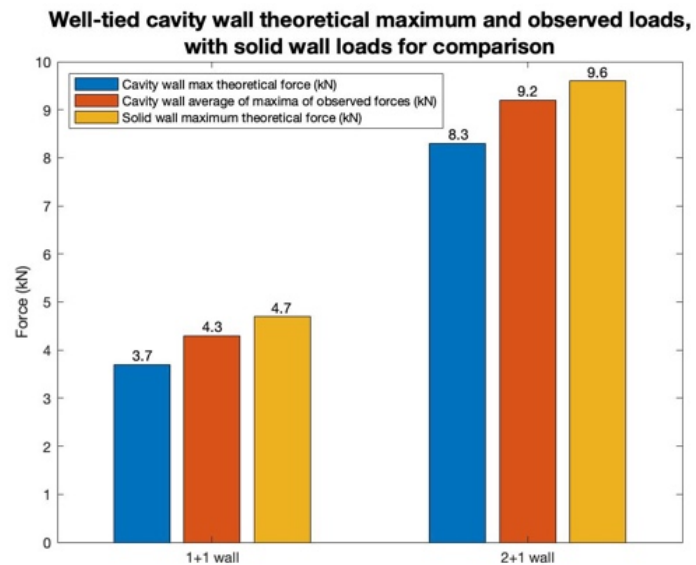## 4.1   Displacement-based analysis of wall capacity

Part C8 of the Guidelines defines the out-of-plane capacity of walls as their ability to reach a limiting displacement before instability causes collapse. Using a modified version of the Parts and Components loading in NZS1170.5, C8 defines the probable deflection response of walls to the lateral loading that is predicted for their location. Analysis has been conducted regarding displacement-based response of well-tied cavity walls using the calculation method given in Part C8, assuming pseudo-rigid behaviour of the walls. This response was compared to the predicted response of solid walls of the same total thickness as the cavity walls. Because cavity walls are lighter than solid walls, they have less rotational inertia and generate less restoring force to resist instability. However, as a result of their lower mass they also incur a lower load than solid walls for a given acceleration. The following analysis shows that the net result of the lower capacity and lower loading of well-tied cavity walls is that if pseudo-rigid behaviour is assumed, they are predicted to perform approximately the same as solid walls. Figure 9a shows a maximum difference of three percentage points in % New Building Standard (NBS) scores of solid walls and well-tied cavity walls at low seismic loading, decreasing to approximately one percentage point at higher loads. Evidence for well-tied cavity walls' displacement-based equivalence to solid walls is found in the deflections they sustained before collapse during experimentation (see Table 1). Based on the attained experimental results, it appears that well-tied cavity walls behaved the same as solid walls for practical purposes, when considered in a displacement paradigm. Therefore, well-tied cavity walls could simply be assessed for displacement capacity by engineers familiar with C8 by assuming that they are solid.

4.2   Force-based analysis of wall capacity

A simplified model of wall behaviour from a force-based perspective is presented in Figure 10a. In the model, the wall cracks into two bodies, which rock cyclically, one atop the other. The model assumes that maximum lateral load capacity occurs at first displacement, when the lever arms are longest between the weight forces in the wall and the pivots. Based on this assumption, a theoretical maximum out-of-plane load capacity for the cavity walls can be calculated. When this theoretical maximum is compared to loads generated by the New Zealand Loadings Standard, a force/capacity ratio can be found which is analogous to the %NBS score generated in the displacement realm. Comparing theoretical capacity/demand ratios of solid walls and cavity walls, analysis shows that they are predicted to perform very similarly, supporting the proposition that if cavity walls are well-tied they can be assessed as solid walls of the same gross thickness (see Figure 9b). When these theoretical loads are compared to experimental results, there is good agreement, with theoretical loads appearing somewhat conservative (see Figure 10b).
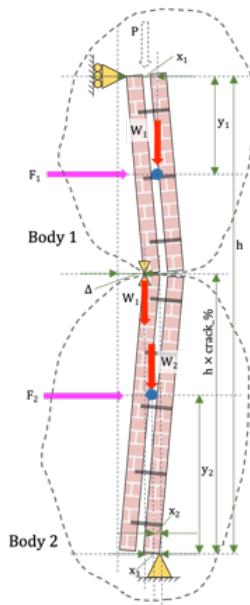
(a) Force-based model of wall system   *(b) Comparison between theoretical loads and observed loads*

Figure 10: Force-based analysis of well-tied cavity walls

Given the variability of materials and seismic loads, it is unrealistic to predict out-of-plane clay brick wall behaviour with high precision. However, the research presented herein shows sufficient equivalence in the force-based and deflection-based paradigms between solid walls and cavity walls to allow well-tied cavity walls to be assessed as solid.

5   CONCLUSIONS

-    The optimal spacing for shear transferring ties in cavity walls used during experimentation was approximately 460 mm horizontal centres, 400 mm vertical centres.

-    This tie spacing corresponds to the state well-tied, which was defined as sufficient tying to ensure that the wall's failure mode was not related to tie performance. Additionally, well-tied cavity walls responded as upper and lower pseudo-rigid bodies.

-    Spacing of ties to result in a well-tied condition do not depend upon the predicted lateral load but instead upon the geometry of a wall, since axial failure modes of the ties were neither predicted nor observed any loading below that which was sufficient to collapse the walls.
-    Theoretical predictions of capacity/demand ratios were similar for well-tied cavity walls when compared to solid walls of the same total thickness. The performance of well-tied cavity walls in the experiments aligned with the theoretical predictions. Thus, well-tied cavity walls can be assessed for out-of-plane capacity as if they were solid walls of the same total thickness.

-    Under a force-based paradigm, capacity/demand ratios have been demonstrated to be similar for solid walls and well-tied cavity walls, using a simple model for assessing peak out-of-plane loading capacity.

-    Experimental results show that the cavity walls resisted loads in excess of the predicted peak theoretical load. Therefore, it is conservative to assume that well-tied cavity walls perform at least as well as solid walls of the same total thickness under a force-based regime.

-    This research is based upon semi-cyclic experiments and further verification using shake-table dynamic testing is underway.

ACKNOWLEDGEMENTS

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