# Retrofitted URM cavity walls experimentally validated and a simplified out-of-plane assessment

*H. Tocher*

Holmes Consulting, Auckland, New Zealand

*N. Slavin*

Department of Architectural Engineering, California State Polytechnic University, San Luis Obispo, California, USA

*U. Maduh, D. Dizhur*

Department of Civil and Environmental Engineering, University of Auckland, New Zealand

## ABSTRACT

How should the post-retrofit capacity of URM cavity walls be assessed? An extensive testing programme was executed to determine the out-of-plane capacity of cavity walls when retrofitted with shear-transferring cavity ties. Walls were tested under semi-cyclic and dynamic shake-table loading. A tie pattern was devised that enabled a cavity wall response corresponding to the model for solid wall behaviour as contained in Part C8 of the MBIE/NZSEE Guidelines. Two different cavity wall typologies were tested (1) two single clay brick leaves separated by an air cavity and (2) two-leaf solid clay brick walls with an external single leaf brick layer separated by an air cavity. Retrofitted walls were able to sustain large mid-height displacements when subjected to out-of-plane loading and had a substantial increase in load-carrying capacity when compared to the walls without shear-transferring ties. Experimentally attained results showed good agreement with theoretical displacement and load-carrying predictions. It was concluded that for practical seismic out-of-plane assessment purposes, walls tied using shear-transferring cavity ties at sufficiently close spacing can be assessed as solid walls of the same gross thickness.

## 1 INTRODUCTION

When retrofitting unreinforced clay brick masonry (URM) walls, engineers are frequently concerned with preventing out-of-plane (OOP) collapse under face loading. URM walls with a cavity present a still-greater challenge for retrofit, as the individual leaves of the wall are often slender and reach an unstable displacement under small loads. It is now commonplace in current engineering practice to improve the performance of cavity walls by retrofitting them with steel ties which span the internal cavity. While the outer leaf is often treated as a non-structural mass which must be restrained using flexible ties in tension, the semi-rigid shear-transferring ties used in these experiments sought to activate the outer leaf as a structural component of the OOP load-resisting system.


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

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

*Figure 1: Wall specimens and details*

Several prior investigations have attempted to verify and measure the OOP performance improvement delivered by cavity ties. Researchers have sought to establish an 'equivalent thickness' value for 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) undertook in-situ testing of cavity walls found in existing buildings, concluding 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) undertook dynamic testing of cavity wall specimens using a shake table. The testing showed that the slender as-built wire or flexible remedial helical 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. By contrast, 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 has been 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 levels of URM buildings and can also be found on upper building levels.

## 2 CAVITY WALL TEST SPECIMENS

The experimental testing programme was conducted using six purpose-built full-scale URM walls (Fig. 1a). The walls were 3000 mm high and 1150 mm wide. Three of the walls had single leaves of brick on either side of the cavity, and were designated "1+1". The other three walls had a double inner leaf and a single outer leaf, and were designated "2+1". The cavity width of all walls was half the thickness of a brick, approximately 50mm. Wall heights, typologies, and cavity thickness were all chosen to approximate an upper storey wall in a typical URM building.

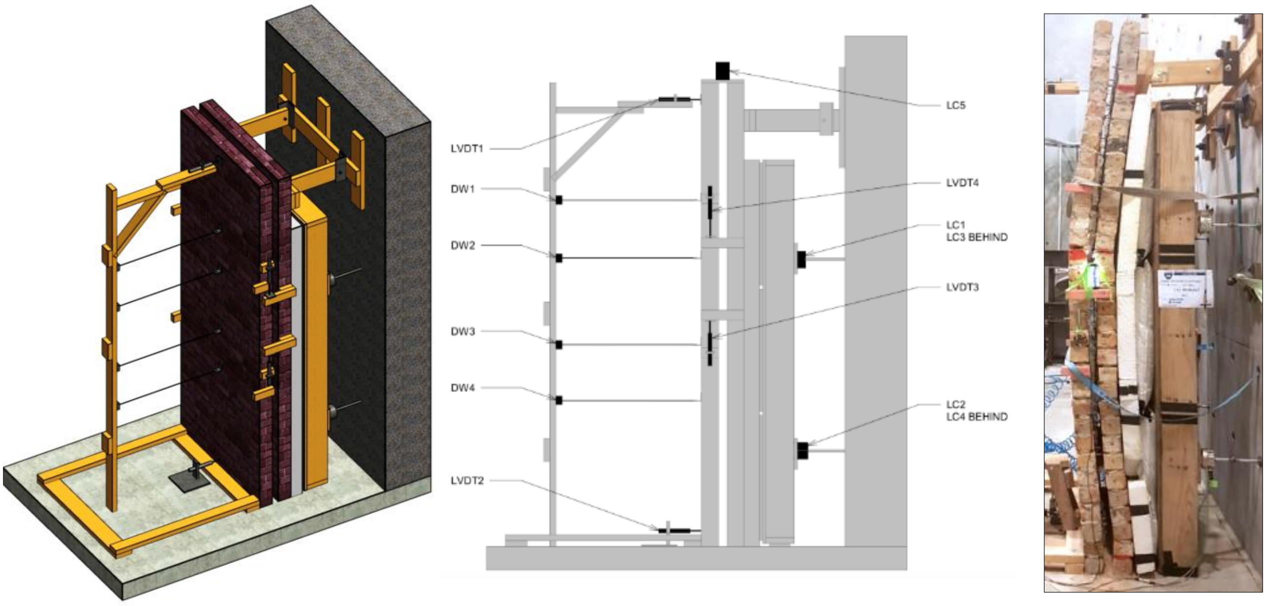
In historic URM cavity wall construction, flexible wire ties were inserted into the walls to connect the leaves across the cavity. Following the methodology employed by Giaretton (2016c), these as-built wire ties were simulated in the specimen walls using 4.0 mm steel wire bent into open hoops (Fig. 1b). Deterioration of the as-built ties was simulated by notching the wire hoops that were used in the specimen. 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, proprietary cavity ties were used to transfer shear and generate composite action between the leaves, constraining both leaves of the cavity wall to move together. The retrofit ties were 6 mm diameter masonry threaded screws with a length of approximately 240 mm and had significantly higher axial and flexural capacity than as-built ties. Spacing of the strengthening ties was investigated by varying it for each retrofit test.

The wall specimens were constructed with bricks 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. 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).

The mean compressive strength of single 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 MBIE/NZSEE Guidelines. The mean compressive strength of sample 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. The mean compressive strength of three-brick 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 (MBIE 2017).

Masonry and mortar compressive strengths are not parameters used in the calculation of the out-of-plane capacity of URM walls under the Guidelines. Although mortar tensile strength is assumed to be zero, the residual tensile capacity of the mortar can contribute to the break-off failure observed in walls with an insufficient number of shear-transferring ties (refer Section 3.1.2). Refer Section 4 for a discussion of tie spacings and failure modes.

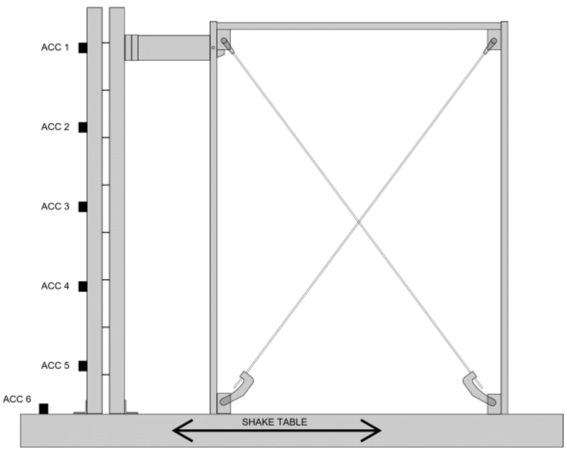



*(a) Isometric schematic of setup* *(b) Side elevation showing instrumentation plan* *(c) Wall under test*

*Figure 2: Semi-cyclic test setup and instrumentation plan*

## 3 TESTING PROGRAMME

The testing programme consisted of two phases, employing different loading methods. The first was semi-cyclic testing, in which walls were loaded unidirectionally using airbags to determine their maximum displacement capacity and to define an optimal spacing for retrofit ties (see (Fig. 2). The shear-transferring ties used in the testing were 6mm diameter 240mm long cavity tie system. This semi-cyclic testing phase was designed to imitate uniform inertial forces of the wall under seismic loading and replicate the model of wall displacement defined in the MBIE/NZSEE Guidelines, thus permitting the assessment of tie-retrofitted cavity wall capacity using Part C8. To provide assurance that the static model in Part C8 gives a conservative assessment of wall capacity under seismic loading, the second phase of testing applied dynamic loading to the specimens using a shake table. In this dynamic phase the failure modes of the walls were closely observed to check if tie failure would limit the walls' ability to rock. Stable rocking with self-centring for both symmetrical and asymmetrical walls was investigated, and an equivalence of loading between semi-cyclic and dynamic regimes was sought using a force-based metric (refer Section 4.1 and 4.2 for a full discussion).




*(a) Side elevation showing instrumentation, shake table, and steel reaction frame*

*(b) Wall under dynamic test*

*Figure 3: Dynamic test setup and instrumentation plan*

In semi-cyclic testing, a system of airbags was used to apply a uniformly distributed load, with the test setup shown in Figure 2a. Each wall specimen 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 mechanical anchoring screws (double threaded 8 mm diameter and 230 mm long, fixing system), 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.

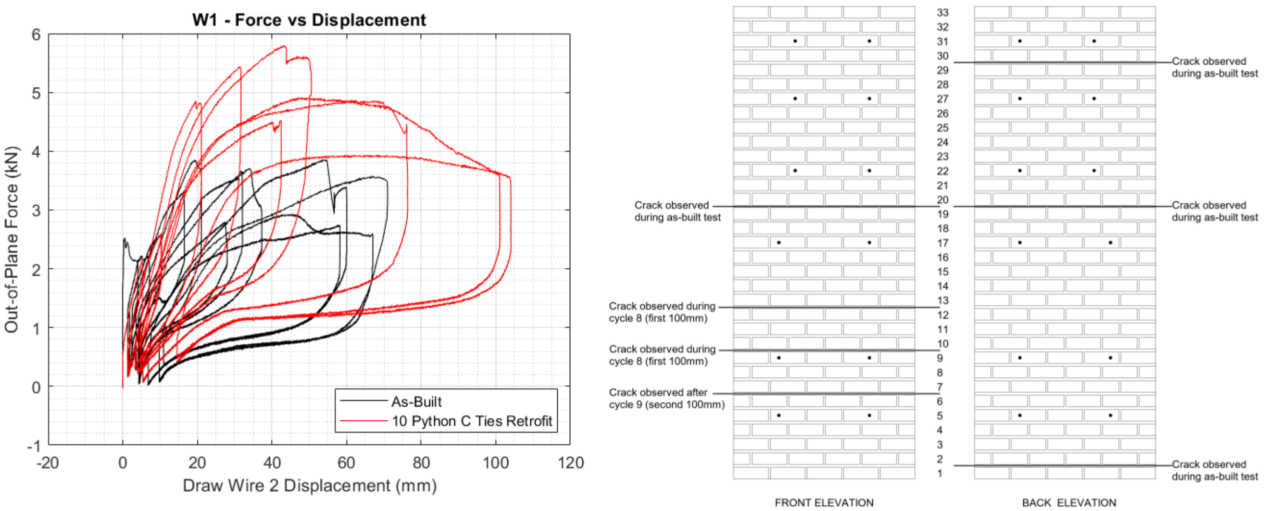
The airbags pushed against the timber reaction frame seen in Figure 2 to the right of the URM wall specimen and were sandwiched between the timber and a layer of polystyrene (shown in white in Figure 2c) to ensure an even distribution of applied pressure. When airbags were inflated, a force was exerted on the wall specimen which was measured by four load cells, located on the opposite side of the timber reaction frame. Loading was applied gradually 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.

In dynamic testing, wall specimens were restrained at the base by steel angles bolted to the bed of the shake table and bearing on the base course of bricks. Walls were tested in an as-built state and with shear-transferring 6mm diameter ties, as for semi-cyclic testing. The top of the wall was connected using mechanical anchoring screws (double threaded 8 mm diameter and 230 mm long fixings) to a timber stringer and timber joists, as with the semi-cyclic specimens. The joists were pinned to a steel frame which was also bolted to the shake table (Fig. 3). The joists allowed rotation of the upper part of the wall, and the steel frame, accelerated by the shake table, provided an approximation of the dynamic response of a surrounding structure. Dynamic tests were instrumented primarily with accelerometers, and displacement measuring devices (which were removed at high levels of excitation due to potential risk of damage). Dynamic displacements, reported below in Section 4, have been measured using video analysis with point-tracking software. The shake table testing used two earthquake records from the February 2011 M6.3 Christchurch earthquake. The first record was from the Christchurch Botanic Gardens (CBGS_S01W), chosen for its repeated high-displacement reversals and the second was from the Lyttelton Port Company (LPCC_S80W), chosen for rapid and violent shaking. The earthquake records were converted to displacement intervals used as inputs to drive the shake table. The control software of the shake table allowed the "playback" of the earthquake to be scaled, with displacement amplitudes modified by a scalar factor. Dynamic testing used an incrementally increasing playback of the earthquake record, starting from 10% amplitude and moving up to 120% - 125% amplitude, at which point the maximum displacement of the shake table (±180 mm) was reached.

## 3.1 Semi-cyclic testing

### 3.1.1 Wall 1 (1+1)

Wall 1 (W1) was a single-single (1+1) cavity wall with gross dimensions 3000 mm by 1150 mm by 275 mm. In the as-built state with only flexible wire ties, a peak deflection of approximately 75 mm at the crack location between courses 19 and 20 was reached after twelve cycles of testing, with a post-crack load fluctuating around 3.5 kN (Fig. 4a). The wall was then retrofitted with ten shear-transferring ties, spaced at approximately 690 mm horizontal, 600 mm vertical. The wall was loaded in cycles with increasing deflections, reaching a peak displacement at the central crack of approximately 112 mm. The total peak force increased from 3.9 kN in the as-built state to 5.8 kN following tie installation. Instruments recording the relative vertical displacement of the inner and outer leaves of the wall recorded an increase in differential movement when lateral mid-height displacement reached approx. 50 mm, indicating that the ties were deforming to accommodate the leaves' relative movement. Differential movement was more noticeable in the block of wall below the crack between courses 19 and 20, and ties in this lower block having undergone some level of plastic deformation.




*(a) W1 force vs draw-wire 2 displacement*

*(b) W1 12 C ties and cracking locations (ties installed from front elevation)*

*Figure 4: W1 Load-displacement relationship, tie layout, cracking*

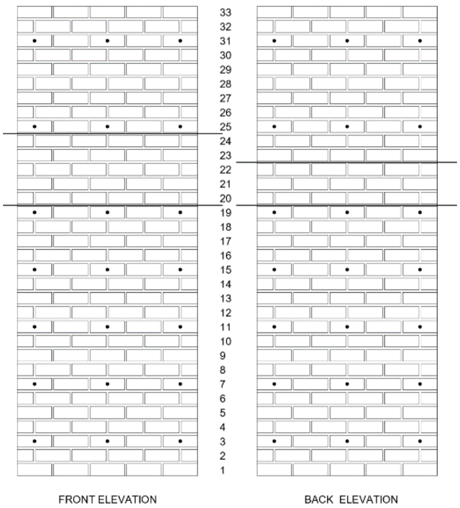
For the final test of W1, two additional ties were added to the wall at course 5 to increase capacity and reduce demand on ties at course 9. The wall withstood deflection to approximately 118 mm at the crack with a reduced peak force of 5.0 kN (14% reduction). Further cracking was observed in the lower part of the wall, with horizontal cracks opening up between courses 6 and 7 and between courses 12 and 13 as illustrated in Figure

4b. The distributed cracking of the wall into multiple blocks is an indication that the adopted tie spacing for W1 resulted in good distribution of forces throughout the wall height.

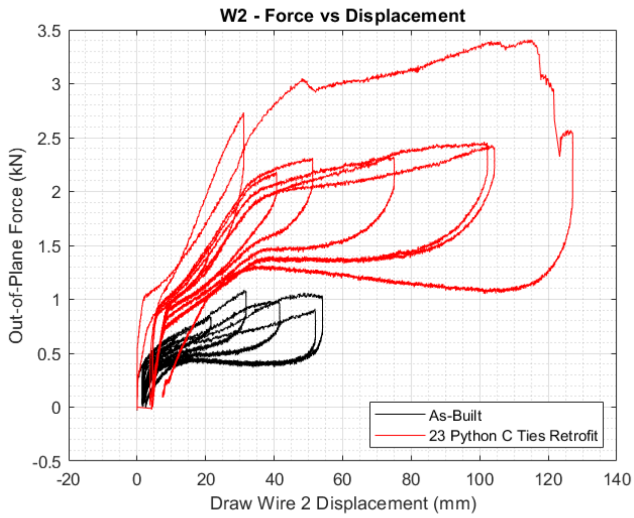
### 3.1.2 Wall 2 (1+1)

Wall 2 (W2) was a single-single (1+1) cavity wall of the same dimensions as W1. Due to the distributed cracking and tie flexure that occurred in W1, W2 was initially retrofitted with a 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 (Fig. 5a). 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. Cracking between tie groups risks a failure caused by the break-off of disconnected masonry courses, leading to collapse either by loss of gravity support or loss of the pivot point for rocking.

Two additional ties were added at course 22 to secure the disconnection upper block of masonry. With the addition of these ties the wall underwent rocking and recovery cycles up to displacements of approximately 100 mm (Fig. 5b). 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.




*(a) W2 21 C ties and cracking locations*




*(b) W2 force vs draw-wire 2 displacement*

*Figure 5: W2 Load-displacement relationship, tie layout, cracking*

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.

### 3.1.3 Wall 3 (2+1)

Wall 3 (W3) was a double-single (2+1) cavity wall with gross dimensions 3000 mm by 1150 mm by 395 mm. Double-single walls are asymmetrical, and it was predicted that their response would be anisotropic. As a result, the semi-cyclic testing programme included turning each 2+1 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” and is shown in Figure 6a. When loading is applied to the single leaf, the orientation is denoted “1+2” (Fig. 6b).

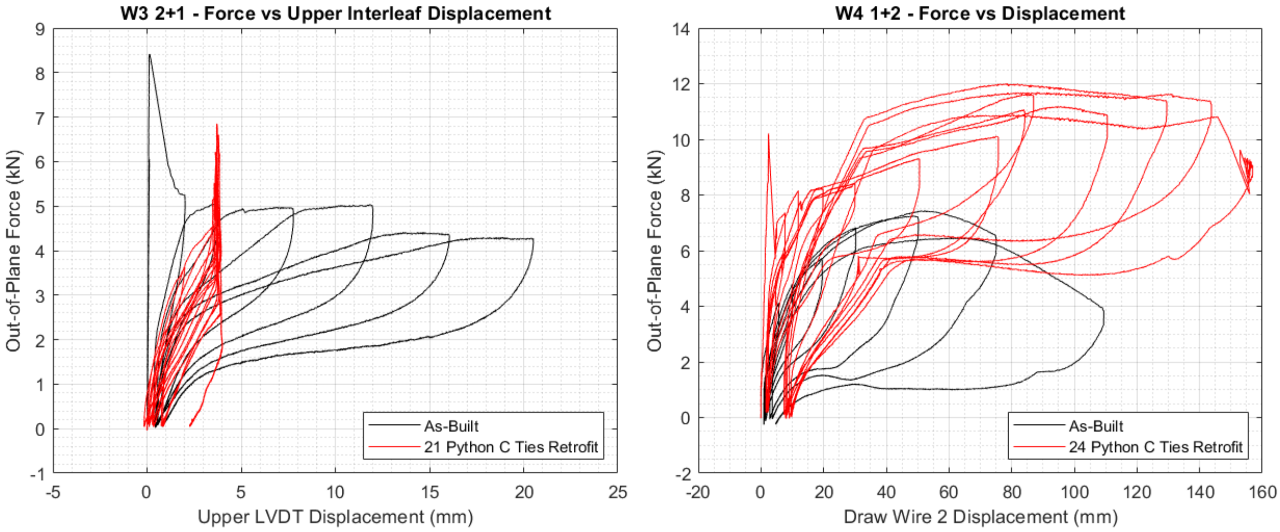
Such turning was not required for dynamic testing where load direction reversal is inherent in the testing method.

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

*Figure 6: Double-single (2+1) walls under loading*

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 an 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 (Fig. 7a). The wall specimen was retrofitted using 21 shear-transferring 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 during the tie retrofit test, suggesting that the cavity wall leaves acted compositely as a semi-rigid body and relative displacement was limited.

Wall 3 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 supported the outer leaf via the retrofit ties (Fig. 6c). To determine if loading direction influenced 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 7: Force vs displacement plots for double-single (2+1) walls*

### 3.1.4 Wall 4 (2+1)

Wall 4 (W4) had the same nominal dimensions and testing setup as W3, except that testing began in the 1+2 orientation. The wall specimen was retrofitted with 24 shear-transferring 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 discussed in Section 4.2. Considering the displacement domain, the wall reached nearly 160 mm at draw wire 2 (Fig. 7b), 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.

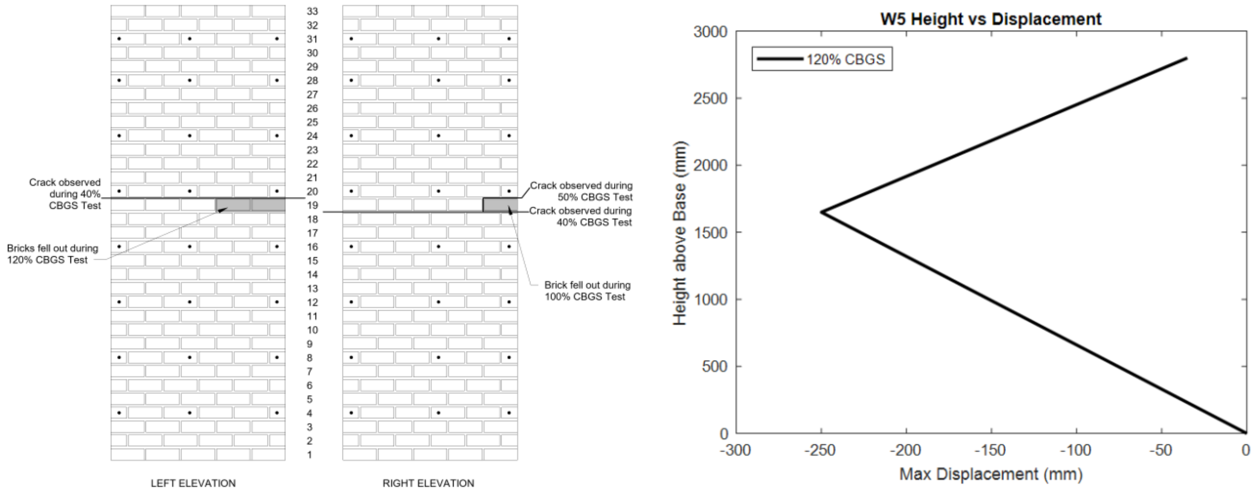
Ties were removed from the wall and displacement cycles commenced for testing in the as-built condition. A displacement of 110 mm was achieved with a rapidly-decreasing total lateral force, suggesting that instability would shortly follow. The wall was rotated to the 2+1 orientation and re-tested, without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

24 shear-transferring 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 retrofitted 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 (Fig. 6d), governed by reaching the geometric instability point of the wall.

## 3.2 Dynamic (shake table) testing

### 3.2.1 Wall 5 (1+1)

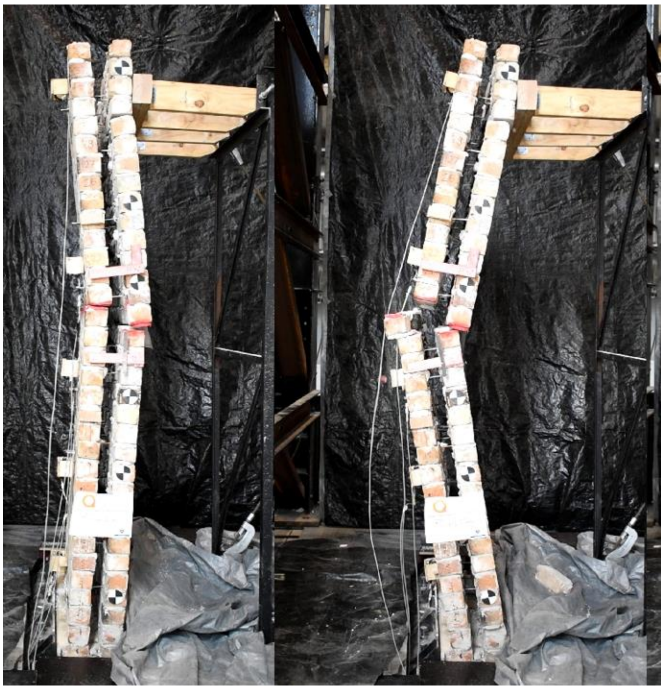
Wall 5 had the same nominal dimensions as W1 and W2, and a mass of 1430 kg. The wall was retrofitted with 24 shear-transferring ties spaced at 460 mm horizontal and 400 mm vertical and tested under ten cycles at increasing amplitude of the Christchurch Botanic Gardens (CBGS) earthquake record described in Section 3.




*(a) W5 1+1 with 24 ties and cracking locations* *(b) W5 1+1 displacement vs wall height*

*Figure 8: W5 dynamic testing cracking patterns and displacements*

Cracking occurred between course 18 and 19 on the outer leaf and course 19 and 20 on the inner leaf, commencing at 20% of the CBGS record and becoming pronounced at 40% (Fig. 8a). Stable rocking behaviour became evident at 40% and continued throughout the testing programme. The observed rocking behaviour closely followed idealised models of rocking. The ties maintained a constant cavity distance between the two leaves of the wall. The wall rocked as two separate portions, and these portions remained essentially planar and rigid as they rocked one atop the other. Under high levels (100% of CBGS record and above) of loading, a secondary crack formed between course 2 and 3, but the lower portion still rocked as a rigid body. Rocking behaviour can be seen in stills taken from testing videos (Fig. 9).

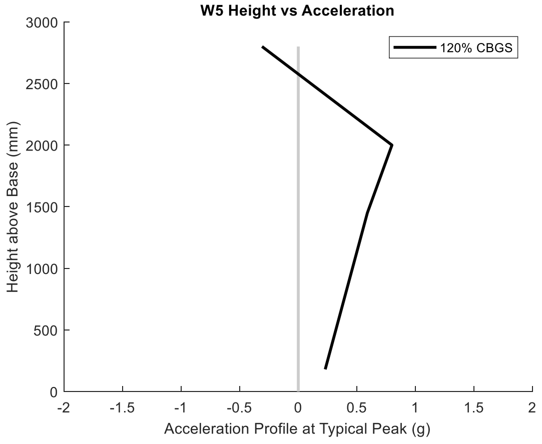


**Scan QR code to view the test video**

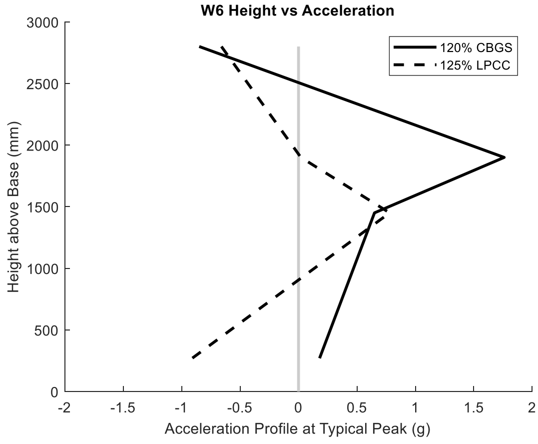
*Figure 9: Wall W5, stills from video record of testing showing rocking behaviour and collapse*

Alongside the classical rocking behaviour, a gradual offset between the upper and lower portions of the wall occurred. After six shaking records of testing, reaching 60% of CBGS, the upper portion of the wall came to rest at a small misalignment from the lower portion (~5mm). The offset remained small during testing to 80% of CBGS. After 100% of CBGS the offset between upper and lower portions increased to around 20 mm, but the wall retained gravity support even with a reduced contact area and was able to self-centre.

After nine shaking records of increasing magnitude, having reached 120% of CBGS, a single brick fell away from the mid-height crack at course 19. The wall's peak lateral displacement at course 19 briefly reached approximately 250 mm relative to the base, with the wall still being able to self-centre. The peak acceleration of the shaking table was 0.7g, and the peak acceleration sustained in this cycle was 1.27g at the top of the wall. A typical acceleration profile for the wall during testing is shown in Figure 10a (note that the plot does not reflect overall maximum recorded accelerations, but provides a typical example of an acceleration profile along the wall height). The wall came to rest at the end of this cycle with the top portion supported by around 20 mm thickness of the lower portion. The wall was then tested with a repeated cycle of 120% of CBGS, at which point the upper portion "walked" off the lower portion and collapsed (Fig. 9). This collapse occurred after the wall was subjected to approximately 120 rocking cycles over 11 different earthquake ground records varying from 5% to 120% of CBGS.




*(a) W5 1+1 typical acceleration profile vs height at a specific point in time*




*(b) W6 2+1 typical acceleration profile vs height at a specific point in time*

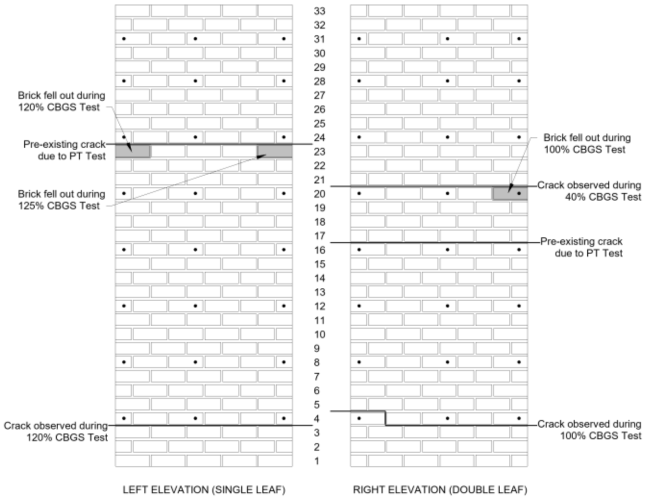
*Figure 10: W5 and W6: typical accelerations profile vs height. Note that the plots do not reflect overall maximum recorded accelerations, but provides a typical example of an acceleration profile along the wall height at a specific point in time.*

### 3.2.2 Wall 6 (2+1)

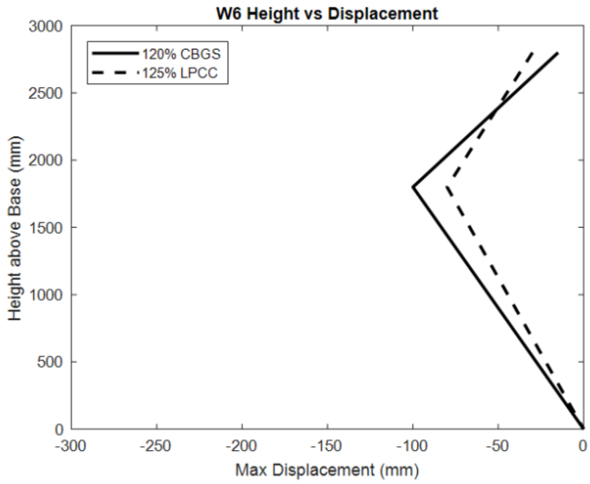
Wall 6 had the same nominal dimensions as W3 and W4 and massed approximately 2150 kg. The wall was retrofitted with 24 shear-transferring ties spaced as for W5, and tested under 16 cycles of the CBGS record and 5 cycles of the LPCC record described in Section 3.

At 40% of the CBGS record, cracking was observed between course 20 and 21 of the double leaf, and between course 23 and 24 of the single leaf. Although the cracking did not occur at the same course in both leaves of the wall, the observed rocking behaviour still agreed with classical models of rocking. Cavity ties maintained a constant separation between inner and outer leaves. Both upper and lower portions rocked as rigid planar bodies.

At 70%, full uplift of the upper portion of the outer leaf was observed. As the testing increased in intensity, some uplift of the double leaf was observed, beginning at 100% of CBGS. This limited uplift may result from the position of the single-leaf crack being several courses above the crack in the double leaf, the crack positions having been predetermined by previous testing of the specimen with post-tensioning. A partial uplift of the double leaf was confirmed when, at 100% CBGS, a single brick was lost from the lower portion of that leaf. The wall self-centred despite the loss of a brick.




(a) *W6 2+1 ties and cracking locations*




(b) *W6 2+1 displacement vs wall height*

*Figure 11: W6 dynamic testing cracking patterns and displacements*

The wall was then subjected to 4 repeated earthquakes of 120% of the CBGS record, then an earthquake of 125% CBGS. Two further bricks were lost from the mid-height region during this testing, as illustrated in Figure 11a. Peak acceleration in these tests was 1.8g at the top of the wall, with a peak mid-height displacement slightly in excess of 100 mm. Peak shaking-table acceleration was 0.67g. Despite the loss of bricks at the contact zone between the upper and lower portions, the wall continued to display rocking behaviour and to self-centre (Fig. 12).

**Scan QR code to view this test video**

*Figure 12: Wall W6: stills from video record of testing showing rocking behaviour*

The wall was then tested with the Lyttelton (LPCC) record, which exhibits fast high-amplitude peaking. The wall continued to rock at the same crack locations noted for CBGS. The record was scaled up through five earthquake cycles reaching 125%. While the measured peak acceleration of 2.3g was higher for the LPCC record than the CBGS record, peak displacement was lower, at around 80 mm. Peak shake table acceleration was 0.96g. A typical acceleration profile for the wall during testing is shown in Figure 10b (note that the plot does not reflect overall maximum recorded accelerations, but provides a typical example of an acceleration profile along the wall height). The wall lost one additional brick from the rocking zone but remained self-centred at rest. Wall W6 did not collapse under dynamic loading despite being subjected to approximately 250 rocking cycles over 21 earthquake records varying from 5% to 125% CBGS and 40% to 125% LPCC.

## 4 RESULTS AND DISCUSSION

The experimental programme aimed to establish whether URM cavity walls retrofitted with ties could be assessed for out-of-plane capacity as solid walls of the same thickness. To assist in examining the data and providing an answer to the research question, the terms "well-tied" and "pseudo-rigid" are proposed.

*Well-tied*. Walls with sufficient cavity ties added to ensure that the failure mode of the wall is not caused by tie failure are considered well-tied. Wall failure will instead occur as a result of a mode associated with the failure of solid walls, for example geometric instability, out-of-plane bed joint shear, or loss of support at the diaphragm. In the testing presented in this paper, the spacing required to create a well-tied condition was found to be approximately 460 mm horizontal (two bricks) and 400 mm vertical (four courses). Wall W1, which had more widely spaced ties, exhibited horizontal cracking in multiple locations. All other walls, retrofitted with the well-tied spacing given above, ultimately failed as a result of overall wall geometrical instability.

After using the results attained from experimental testing to determine the optimal spacing for well-tied walls, experimental results were compared to first-principles calculations of the force required to initiate various modes of tie failure, with the following observations:

- First-principles calculations and further investigations showed that tie failure modes such as: pull-out and buckling are sufficiently far up the force hierarchy that they are extremely unlikely to occur before solid-wall failures like instability.
- For rocking to be sustained, tie flexure is the critical tie failure mode that needs to be prevented.
- The critical loading case for flexure is at the initiation of uplift when wall weight is directed perpendicular to the ties' central axis (Fig. 14a). The magnitude of this demand is related to the size and weight of the wall, not to the out-of-plane load applied to the wall.
- Any significant lateral loading will cause uplift of the outer leaf above the mid-height crack and impose maximum demand upon the ties. Thus, the number of ties required to create a well-tied condition for out-of-plane loading is the number required to allow rocking to occur through uplift of the outer leaf.

*Pseudo-rigid*. Well-tied cavity walls perform as solid walls through rocking of the upper and lower parts of the wall. Rather than each individual leaf cracking and rocking separately, it is proposed that the entire well-tied wall cracks horizontally at approximately two-thirds of its height. After cracking, the upper portion rocks upon the lower portion, with inner and outer leaves taking turns to form the pivot point and to be uplifted and fully supported by the ties (Fig. 13). For predictions to be made about the force capacity of well-tied walls exhibiting rocking behaviour, and for such rocking to be reliable under multiple cycles of reversing loads, the position of the centre of mass of the rocking blocks must be predictable. Predictable centres of mass exists if the ties hold the inner and outer leaves near-rigid during rocking, a condition defined herein as pseudo-rigid.

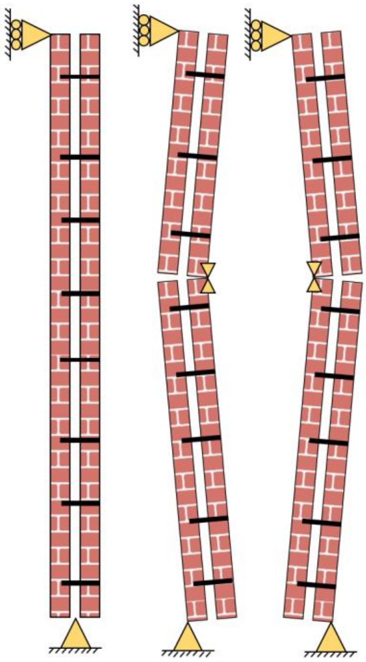


*Figure 13: Idealised model of pseudo-rigid rocking of well-tied cavity walls*

Pseudo-rigid rocking allows the outer leaves of cavity walls to serve a structural purpose. Note that a distinction is made here from the usual assumption that the outer leaf serves only as ancillary mass, doing no structural work, and that the purpose of cavity ties is to anchor this ancillary mass back into the inner (structural) wall. Instead, the rigid ties allow the outer wall to function structurally and contribute restoring moment to the system across the lever arm created by the shear transferring ties.

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

All cavity walls in this testing programme demonstrated pseudo-rigid rocking behaviour. The upper portions of each specimen demonstrated pure rocking, whereas lower portions frequently adopted a more curved shape, especially in semi-cyclic testing. The lack of rigidity in the lower block likely results from the overburden from the upper block, generating a partial flexural response in the ties. Despite a less classical rocking response of the lower portions, the self-centring behaviour inherent in rocking was observed in all specimens, suggesting that the practical difference between observed and theorised behaviour is small.

### 4.1 Displacement-based assessment of OOP capacity of well-tied cavity walls

Assessment of the out-of-plane capacity of URM walls under Part C8 of the MBIE/NZSEE Guidelines uses a displacement-based metric to quantify performance. Semi-cyclic airbag testing was chosen to simulate the loading conditions inherent in the Part C8 model and establish the displacement capacity of well-tied cavity walls. Using C8, predictions were calculated for the instability displacement capacity $\Delta_i$ and maximum usable displacement capacity $\Delta_m$ of solid URM walls having the same total thickness as the cavity walls tested in the experiments. A typical 1+1 cavity wall has a total thickness of approx. 275 mm. Although typical solid brick walls would not have a thickness of 275 mm because it is not a multiple of common brick sizes, the C8 methodology allows for capacity prediction of walls of any thickness.

Semi-cyclic testing showed that well-tied cavity walls were able to achieve the displacement levels predicted for solid walls. All walls retrofitted with cavity ties were displaced well past the predicted maximum usable displacement and were still able to self-centre. The 2+1 walls W3 and W4, which were tested to collapse in a retrofitted state, were able to exceed the predicted maximum usable displacement, and did not collapse until they reached the approximate instability displacement predicted for solid walls (Table 1). Peak displacements of retrofitted walls increased markedly over the displacements of walls in an as-built condition.

*Table 1: Displacement of well-tied walls compared to Part C8 of the MBIE/NZSEE Guidelines displacement predictions*

| **Wall type (total thickness)** | **Measured displacement with self-centring (mm)** | **Predicted maximum usable displacement for solid wall of same total thickness** | **Measured displacement before collapse** | **Predicted displacement at collapse** |
|---|---|---|---|---|
| 1+1 with flexible ties | 65 mm | 30 mm | 90 mm | 50 mm |
| 1+1 (275 mm) | >149 mm | 115 mm | Not tested | 193 mm |
| 2+1 (395 mm) | >193 mm | 178 mm | 350 mm | 297 mm |

The goal of this research is to allow engineers to make conservative assessments of the capacity of retrofitted cavity walls using Part C8 of the MBIE/NZSEE Guidelines. The semi-cyclic testing and results shown in Table 1 offer the most direct evidence in support of this assessment. Nevertheless, engineers and non-engineers alike would recognise that semi-cyclic unidirectional loading is not the same as the seismic loading generated by an earthquake. For this reason, dynamic testing of cavity walls was also undertaken as part of the testing programme, and measurements of peak wall displacement during this testing were gathered using video tracking software. Both 1+1 and 2+1 walls demonstrated the ability to withstand high levels of displacement under dynamic loading. Wall W5 (1+1) reached approximately 90 mm mid-height displacement in several cycles, and in the penultimate test briefly reached approximately 250 mm of mid-height displacement and still self-centred. This observed displacement well exceeds Part C8 of the MBIE/NZSEE Guidelines predictions for both usable and collapse displacement of a solid wall of the same thickness.

Wall W6 reached approximately 110 mm mid-height displacement in several tests without collapse, with the maximum displacement capacity of the shake-table deployed. While this displacement is not as high as the predicted maximum displacement under C8, the wall did not collapse under dynamic testing, and had the ability to rock further. Under seismic loading, which rapidly reverses direction, peak displacements of the order of those shown in semi-cyclic testing may not have time to occur. In addition to displacement-based comparison of dynamic and semi-cyclic testing, equivalence was shown using a force-based approach, as discussed in the following section.

The primary outcome of the dynamic testing from a displacement perspective was the demonstration that well-tied walls are able to exhibit pseudo-rigid rocking behaviour with cyclical uplift of both inner and outer leaves, with self-centring and without failure modes associated with the ties.

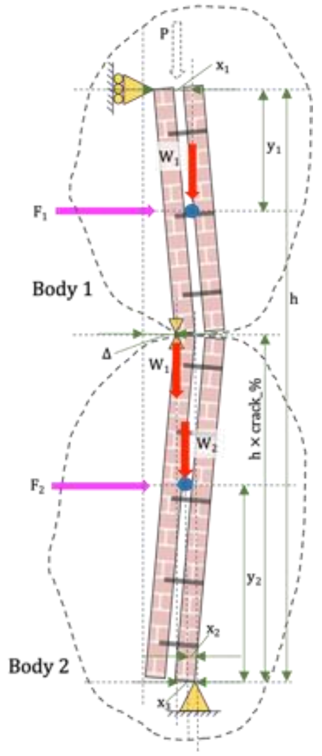
A simplified model of the well-tied wall system was devised to determine the theoretical force capacity of walls, and is presented in Figure 14. The model is closely related to that given in Part C8, and uses restoring moments derived from the wall sections' weights to resist lateral loading, with the peak load capacity occurring at the instant of uplift of the outer wall section. The predicted loads in this model have good agreement with loads derived using NZS 1170.5 Section 8 (Parts and Components).

Cavity walls are modelled as attracting a lower load than solid walls of the same total thickness, as a result of their lower mass. As a concomitant, the walls also have less restoring moment to resist loading. The net result is a minor difference between solid and cavity walls, with solid walls having a slightly higher predicted maximum force capacity.

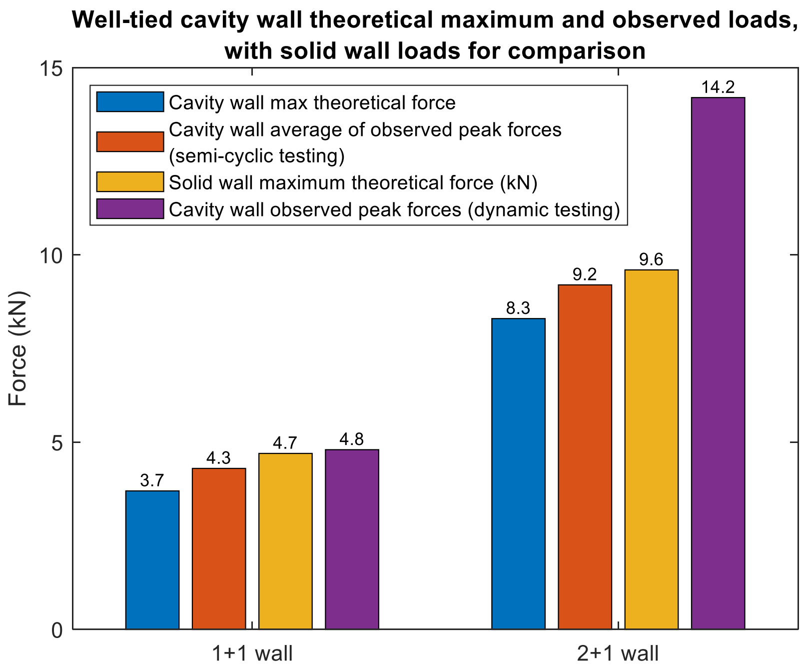
Under semi-cyclic testing, the specimens were able to withstand peak loading greater than the predicted maximum for cavity walls, but lower than the maximum predicted for solid walls. Substantial variation in the force-based capacity of as-built walls resulted from material variability, significant cracking damage, and restraint conditions. However, each wall in a tie-retrofitted condition had increased force capacity over the same wall specimen in its as-built state.

2+1 walls were able to sustain greater lateral force when the double leaf was the outer leaf, compared to testing in the other orientation with the single leaf on the outside. This accords with the simplified model, in 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 double-leaf-outward orientation, perhaps because the central pivot must be supported on a single leaf.

The magnitude of peak loading was found for dynamic testing using the data from the accelerometers. Each timestep of the data was checked to find the peak sum acceleration for any single instant of a test cycle. The accelerations recorded at each level of the wall were then multiplied by the tributary wall mass and the resulting forces summed to provide a total loading.




*(a) Force-based model of wall system*




*(b) Comparison between theoretical loads and observed loads*

*Figure 14: Well-tied, pseudo-rigid cavity wall rocking model*

Under dynamic testing, the peak load sustained without collapse by the 1+1 wall W5 was approximately 4.8 kN, slightly over 110% of the average peak load applied to the 1+1 wall specimens W1 and W2 under semi-cyclic testing, and approximately 103% of the theoretical maximum force for solid walls of the same total thickness as 1+1 walls found using the simplified model.

The 2+1 wall W6 sustained a peak force of approximately 14.2 kN, over 150% of the average peak force applied to W3 and W4 in semi-cyclic testing, and approximately 148% of the theoretical maximum force under the simplified model. These values suggest an oversimplification of the force-based model used to calculate theoretical maxima, but also show the robustness of the well-tied walls under severe dynamic loading. The loads applied by dynamic testing exceeded the loading in semi-cyclic testing, in which walls outperformed the maximum usable displacement predictions of Part C8. Such robustness provides increased confidence in the hypothesis that well-tied cavity walls can be assessed as solid under Part C8 for the purposes of determining post-retrofit capacity.

## 5 CONCLUSIONS

The optimum cavity shear transferring tie spacing for the tested wall specimens was 460 mm (two bricks) horizontally and 400 mm (four courses) vertically. This spacing corresponds to the “well-tied” state, where ties are spaced closely enough to enable uplift of the opposite leaf of the wall through multiple cycles of rocking and in which failure of the wall does not result from tie related failure. The investigations showed that tie failure modes like pull-out and buckling are sufficiently far up the force hierarchy that they are extremely unlikely to occur before solid-wall failures like instability.

Well-tied cavity walls tested semi-cyclically demonstrated the ability to exceed predictions for maximum usable displacement for fully solid walls made using Part C8 of the MBIE/NZSEE Guidelines. Walls that were tested to collapse reached the point of geometric instability.

The ties permitted the tested walls to undergo pseudo-rigid rocking behaviour utilising the full thickness of the wall section, with outer leaves acting as structural components, not simply as ancillary mass. Under dynamic testing, walls sustained high displacements and demonstrated uplift of the outer leaf.

Predictions for maximum usable displacement capacity made using Part C8 of the MBIE/NZSEE Guidelines were conservative for both the symmetrical (1+1) and asymmetrical (2+1) cavity wall specimens.

Under a force paradigm, walls tested semi-cyclically were able to sustain loads in excess of predicted maxima for cavity walls using a simplified model of force capacity. Under dynamic loading, both the 1+1 and 2+1 walls sustained loads higher than the average peak loads used in semi-cyclic testing, without collapse.

Displacement predictions made using Part C8 are conservative for the measured displacement capacity of well-tied walls, and walls have been shown through both semi-cyclic and dynamic testing to withstand predicted forces without collapse. It is therefore proposed that for the purposes of practical engineering assessment, well-tied cavity walls can be assessed using Part C8 of the MBIE/NZSEE Guidelines for retrofitted out-of-plane capacity and treated as solid wall of the same thickness.

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