Out-of-Plane In-Situ Testing of Masonry Cavity Walls in As-Built and Improved Conditions

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ABSTRACT

Extensive research has been performed previously on assessing the out-of-plane (OOP) performance of unreinforced masonry (URM) walls and retrofitting URM load-bearing and infill walls for OOP capacity. However, little research has been performed within New Zealand pertaining to clay brick masonry walls with cavities, despite their prominence in the building population. Hence, further research was pursued with an emphasis on retrofitting URM cavity walls so as to form composite behaviour efficiently. This research was based on an experimental testing approach wherein walls were loaded OOP using inflatable air bags. Testing was performed on ten URM cavity walls in two separate buildings.

The intended outcomes of the research reported herein were as follows:

- Determine the behaviour of cavity walls in vertical flexure when bordered by rigid moment-resisting reinforced concrete (RC) frames;
- Determine the improvement in drift levels in cavity walls prior to loss in strength and prior to collapse, using a variety of cavity wall ties at different spacings;
- Determine which cavity ties are most effective in improving OOP cavity wall performance;
- Determine how the load-based capacities of the test walls correspond to design basis earthquake demands in Auckland (area of moderate seismicity) and in Hastings, New Zealand (area of high seismicity); and
- Determine what effective solid wall thickness should be assumed for cavity walls with various retrofit ties conditions for use in existing models regarding the OOP flexure of URM walls.

Keywords: unreinforced masonry, earthquakes, out-of-plane, infill walls, cavity walls, masonry anchors, cavity wall ties

1. INTRODUCTION

The earthquake vulnerability of buildings constructed using conventional British architecture and unreinforced clay brick masonry (URM) construction prior to the introduction of modern seismic loading standards is well-known in New Zealand based on observations from historical earthquakes (Brodie and Harris 1933; Davey and Blaikie 2010; Ingham and Griffith

2011a). A high proportion of such structures in existence have not been retrofitted to resist seismic forces. The performance of seismically deficient buildings (particularly of clay brick load-bearing URM construction) during the 2010 – 2011 Canterbury earthquakes was the most recent example of the vulnerability of URM construction when subjected to seismic lateral loads (Dizhur et. al. 2010, 2011; Ingham and Griffith 2011b, 2011c; Cooper et al. 2012). However, little research has been performed within New Zealand on URM cavity walls, despite the prominence of this construction type in the building population (Walsh et al. 2014). In order to meet the demand from the New Zealand engineering community regarding knowledge of the OOP behaviour of URM cavity walls, researchers at the University of Auckland physically tested walls in two different buildings utilising an approach consistent with the testing procedures implemented by Derakhshan et al. (2013) and Dizhur (2013). Application of lateral loads using airbags to simulate OOP wall loads was used to determine the OOP capacity of URM cavity walls.

The test walls were isolated into vertically spanning panels (i.e., restraints at the top and bottom only), each with a base length of 1.2 m. The geometries and restraint conditions of the tested walls are summarised in Table 1. Note that the total thickness without ("w/o") including the cavity width, b_{wythes} , represents the thickness of two bricks and is considered for purposes of determining wall weights. The retrofit cavity ties used in this study included both mechanical and adhesive ties and were installed on each test specimen along two vertical lines separated by approximately 600 mm horizontally. Cavities between brick wythes were typically 50 mm.

Table 1: Geometry and cavity tie detailing of test walls

Test ID	**Top edge restraint**	**Test height, h_{test} (mm)**	**Full in situ height, *h* (mm)**	**b_{wythes} (mm)***	**Cavity ties @ vertical spacing**	**Total # of tie legs, *n***
Auc-W1A	Propped	2700	3020	215	Existing "Warrington" wire ties, 4 mm dia.	28
Auc-W1B	Propped	2700	3020	215	Mech.12 mm dia. @ 330 mm	16
Auc-W2	Propped	2700	3020	215	Adhesive 6 mm dia. @ 330 mm	16
Auc-W3	Propped	2700	3020	215	Mech. 8 mm dia. @ 330 mm	16
Has-W1	Rigid	3750	3950	225	Mech.12 mm dia. @ 338 mm	20
Has-W2	Rigid	3750	3950	225	Mech. 12 mm dia. @ 611 mm	10
Has-W3	Propped	3570	3950	225	Mech. 12 mm dia. @ 338 mm	20
Has-W4	Propped	3570	3950	225	Mech.12 mm dia. @ 611 mm	10
Has-W5	Rigid	3750	3950	225	Existing "Warrington" wire ties, 4 mm dia.	34
Has-W6	Rigid	3750	3950	225	Mech. 12 mm dia. @ 152 mm	26

*sum of thickness of two brick wythes excluding cavity

All test walls listed in Table 1 rested on a reinforced concrete (RC) slab. Walls having a rigid top edge restraint consisting of a RC slab or beam were assumed to be subjected to arching action during loading. The effect of such arching action on solid masonry infill walls has been considered by McDowell et al. (1956) and Angel et al. (1994). Walls having only a top edge lateral restraint consisting of timber (i.e., "propped") were assumed to have no arching action. The strength and displacement capacities of load-bearing solid masonry walls spanning vertically have been considered previously by many researchers, most notably Derakhshan et al. (2014). The cavity walls were tested in one-way "vertical flexure" only so that effective solid-wall thicknesses could be derived from the existing models.

2. PREPARATION OF TEST WALLS

The first test building was located at 151-165 Victoria Street West, Auckland and was tested prior to its demolition for purposes of site redevelopment. The building components that were tested were originally constructed in 1958. Test walls Auc-W1, Auc-W2, and Auc-W3 were prepared by cutting through both leaves of a larger wall panel at such spacing as to produce three 1200 mm wide x 2700 mm tall sections of masonry walls separated by approximately 50 mm along the wall base length from one another [see Figure 1(a)] and with the in situ wire cavity ties intact. In situ 4 mm diameter “Warrington” wire cavity ties twisted in a “figure eight” configuration were generally spaced at 900 mm horizontally and 320 mm vertically in a staggered arrangement. The top few rows of bricks were removed from all three test walls in order to separate the tops of the walls from the RC beam above. A timber support was added at the top of the walls in order to restrain horizontal translation while permitting rotation. This arrangement of boundary conditions more closely resembles URM load-bearing walls that extend into timber-framed roofs. Auc-W1 was tested twice – initially with only the in situ ties and secondly with mechanical cavity ties added. Walls Auc-W2 and Auc-W3 were retrofitted prior to testing with adhesive ties and alternative mechanical ties, respectively [see Figure 1(b)].

The second test building was located at 409-429 Heretaunga Street West, Hastings. The original part of the building containing the test walls was constructed between 1931 and 1950. Preparation of all walls in the Hastings building was performed by cutting through both leaves of the wall at such spacing as to initially produce six 1200 mm wide by 3750 mm tall sections of two-leaf masonry cavity walls separated by approximately 50 mm and with in situ cavity ties intact [see Figure 1(c)]. All walls were tested with the in situ ties left in place, the spacings of which were similar to those measured in the building in Auckland. Test wall Has-W5 was tested with only in situ ties, while all other test walls were retrofitted using 12 mm diameter by 230 mm long mechanical ties at different vertical spacings. As with the walls tested in the Auckland building, the top two rows of bricks and mortar were removed in test walls Has-W3 and Has-W4 (reducing the test heights of these walls to 3570 mm). All other walls were tested with the top of the walls being in their original condition, restrained by the relatively rigid RC beam above and, as a result, subjected to arching action.

(a) Instrumentation framing at Auc-W3 after Auc-W1 and Auc-W2 have been tested and cracked

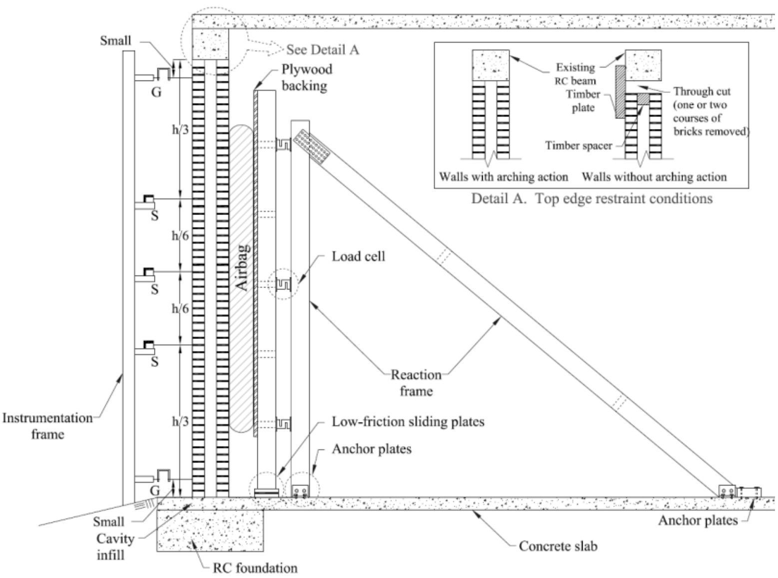
(b) The three retrofit cavity ties used in Auc-W1B, Auc-W2, and Auc-W3, respectively (left to right)

(c) Test reaction frame after testing Has-W1 to complete collapse and prior to testing Has-W2

Figure 1: Test walls and cavity ties

3. TEST SETUP AND INSTRUMENTATION

Loading was applied to the test walls by gradually inflating one vinyl airbag located near the centre of each wall, creating a loaded area approximately 1150 mm horizontally by 2050 mm vertically. The airbag was positioned in a gap of 25-35 mm between the test walls and a plywood-backed frame panel. The plywood backing consisted of an assemblage of plywood sheets and timber frames (see Figure 2). The applied load from the airbag was transferred from the plywood backing to the braced reaction frame using six s-shaped load cells (each with a capacity of 10 kN) which provided the primary source of horizontal stability to the plywood-backed frame panel. To ensure that the entire load was transferred through the load cells and not resisted by bearing friction, the plywood-backed frame panel rested on greased steel or Teflon^{TM} plates to allow the panel to slide with minimal frictional resistance. The neighbouring braced reaction frame consisted of vertical and diagonal timber members that were screw fixed into the concrete floor slab. The total lateral load at any given time was calculated as the summation of the force recorded by all load cells.



(a) Schematic of OOP test bracing (right) and displacement instrumentation (left) [h = test wall height, S = string potentiometer, G = strain gauge]

(b) Profile of OOP test bracing (right) and displacement instrumentation frame (left)

Figure 2: Components of reaction and instrumentation frames for OOP loading of test walls

The instrumentation used to measure the OOP displacement of each test wall was generally placed on an isolated frame located on the opposite side of the wall to where loading was applied (see Figure 2). The instrumentation frame supported multiple strain gauges (G) and string potentiometers (S) during any single test. Highly sensitive digital callipers were also placed at critical locations for redundancy. A high-speed data acquisition (DAQ) system with multiple channels was used to record the test measurements at a frequency of at least 10 Hz. All test walls (with the exception of Auc-W1A) were able to be tested to complete collapse.

4. CAVITY TIE PULL-OUT STRENGTH

The cavity ties used to retrofit test walls Auc-W1B, Auc-W2, and Auc-W3 were tested in isolated pull-out tests. The results are summarised in Table 2. Note that the 12 mm diameter mechanical tie type, which permitted test wall Auc-W1B to outperform its counterparts (as will be described in a later section), also was found to have the highest isolated pull-out strength. However, cavity tie pull-out was not visually observed to limit the test wall OOP capacities.

Table 2: Summary of pull-out testing results from the Auckland building

Test wall	Cavity tie type	Tie diameter / length (mm)	Failure mode	Mean load capacity (kN)
Auc-W1B	Mechanical	12 / 230	brick conical breakout	15.2
Auc-W2	Adhesive	6 / 230	tie steel yield	8.7
Auc-W3	Mechanical	8 / 220	pull-out	1.8

5. WALL MATERIAL PROPERTIES

Samples of brick and mortar were randomly extracted from the test walls in the Auckland and Hastings buildings. A summary of the various material tests, relevant standards, and results is included in Table 3. Where appropriate samples for particular tests were not available, values were determined from empirically-based formulae (AIUMBER 2012; Almesfer et al. 2014). Characteristic "lower bound" material strength values may be derived by subtracting one standard deviation from the mean. Assuming that a normal distribution applies to the samples, 84% of the strengths for individual tested samples should be higher than these "lower bound" values, in this case. Note that the relative strength of the bricks and mortar in the Hastings building are unusual, as compared to historical brick masonry construction in New Zealand which is expected to be comprised of "strong" brick and "weak" mortar (Almesfer et al. 2014; Lumantarna et al. 2014).

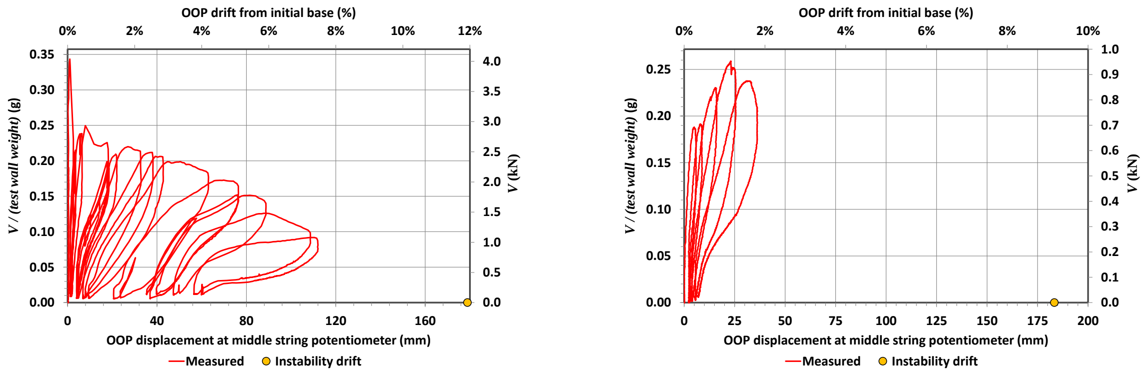
Table 3: Summary of measured and calculated masonry material characteristics

Material characteristic	Associated standards and references for testing and processing results	Number of samples / mean / sample standard deviation (MPa unless noted otherwise)	
		Auckland building	Hastings building
Mortar compression strength, f'_j	Valek and Veiga (2005), ASTM C1314-11a (2011) and Lumantarna et al. (2014)	5 / 13.9 / 1.2	8 / 27.9 / 7.2
Brick compression strength, f'_b	ASTM C67-11 (2011)	5 / 35.5 / 2.9	8 / 11.2 / 1.9
Stacked masonry prism bond rupture strength, f'_{fb}	ASTM C1072-11 (2011)	0.42*	4 / 0.28 / 0.10
Stacked masonry prism compression strength, f'_m	ASTM C1314-11a (2011)	2 / 9.4 / 2.8	3 / 8.2 / 1.6
Stacked masonry prism elastic stiffness, E_m	ASTM C1314-11a (2011)	2 / 3504 / 1389	3 / 5356 / 1775
Brick rupture strength (modulus of rupture), f'_{mr}	ASTM C67-11 (2011)	4 / 3.6 / 0.85	1.3*
Stacked masonry prism density, ρ_m (kg/m^3)	ASTM C1314-11a (2011)	3 / 1720 / 51.7	3 / 1659 / 15.9

* Determined by equation (AIUMBER 2012, Almesfer et al. 2014)

6. WALL TEST RESULTS AND DISCUSSION

All test walls were loaded semi-cyclically at a quasi-static loading rate. The maximum in-test g-force value for each wall was determined by dividing the maximum total test load by the weight of the test wall, as shown in Figure 3. Where walls were able to be tested to complete collapse, the instability drift was measured using photogrammetry.



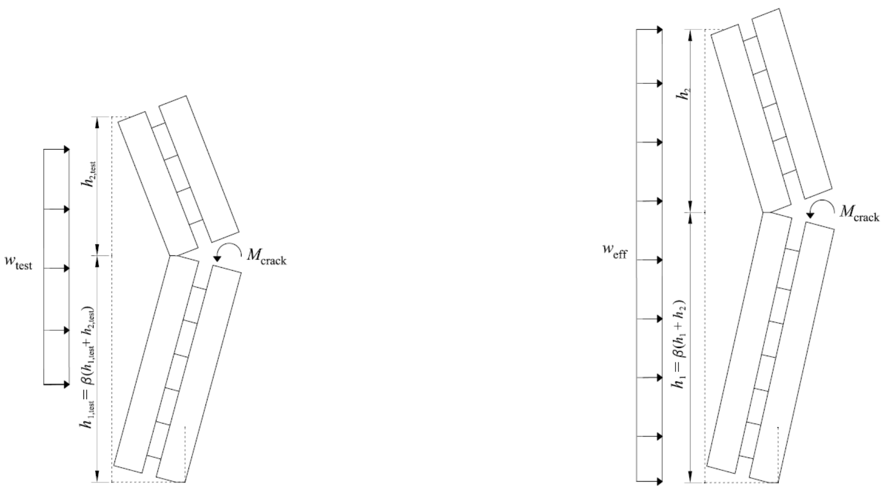
(a) Auc-W2 (b) Has-W4

Figure 3: Select load-displacement responses (displacement measured at mid-height)

The measured partially distributed maximum test load, w_{test} , for each wall specimen from the test condition shown in Figure 4(a) was converted to an effective uniformly distributed maximum earthquake load over the full-height wall, w_{eff} , as shown in Figure 4(b) in order to account for the following variables:

- The loaded wall area was always shorter than the full wall height (i.e., the area loaded by airbag was only 2050 mm vertically);
- The loaded wall area was not always centred on the wall height; and
- The test wall heights were shorter than the in situ wall heights.

The results of this conversion from the test scenario to the assessment scenario for all of the walls tested are summarised in Table 4. The "pivot point" in the analytical conversion from the condition shown in Figure 4(a) to the condition shown Figure 4(b) is the maximum flexural capacity of each wall at the cross-section corresponding to the primary horizontal crack, M_{crack} , occurring at a height above the wall base represented by the crack height ratio, β , which was maintained as equal for both conditions. All walls were considered to be simply supported for this conversion. This analytical conversion approach produces similar results to a conversion procedure incorporating external virtual work with a constant unit displacement occurring at the crack location, such as that utilised by Angel et al. (1994). However, the analytical conversion procedure proposed here more readily accommodates varying wall heights (i.e., test wall height versus in situ wall height) and eccentric loading locations.



(a) Test condition (b) Effective in situ seismic condition

Figure 4: Conversion of measured loads from test condition to effective seismic condition

Table 4: Summary of conversion of measured loads from test condition to effective seismic condition

Test ID	Top edge restraint Lateral only	**Test walls at partial, saw-cut height and with partially distributed test loads**					**Effective full-height walls with full-height, uniformly effective loads**			
		Max post-crack load (kN)	Partial-height uniformly distributed load, w_{test} (kN/m)	Crack height above bottom, $h_{1,test}$ (mm)	Crack height ratio, β	Max flexural moment at crack height, M_{crack} (kNm/m)	Full in situ height of wall, $h_1 + h_2$ (mm)	Effective full-height uniformly distributed load, w_{eff} (kN/m)	Weight of wall full-height (kN)	Effective force-based capacity of full-height wall (g)
Auc-W1A	Propped	2.6	1.27	1891	0.70	0.75	3020	0.94	13.15	0.22
Auc-W1B	Propped	5.5	2.69	1891	0.70	1.60	3020	2.00	13.15	0.46
Auc-W2	Propped	2.8	1.37	1970	0.73	0.76	3020	1.01	13.15	0.23
Auc-W3	Propped	3.4	1.66	1733	0.64	1.08	3020	1.24	13.15	0.28
Has-W1	Rigid	16.4	8.02	1892	0.50	9.33	3950	5.74	17.36	1.31
Has-W2	Rigid	11.9	5.80	1789	0.48	6.73	3950	4.15	17.36	0.94
Has-W3	Propped	4.5	2.19	2514	0.70	1.94	3950	1.43	17.36	0.33
Has-W4	Propped	4.3	2.09	2514	0.70	1.85	3950	1.37	17.36	0.31
Has-W5	Rigid	10.2	4.96	2217	0.59	5.53	3950	3.52	17.36	0.80
Has-W6	Rigid	21.1	10.28	1684	0.45	11.81	3950	7.34	17.36	1.67

7. COMPARISON OF WALL STRENGTHS TO EXPECTED DEMANDS

The relative hazard factor "Z" utilised by the New Zealand loadings standard (NZS 1170.5:2004) is approximately equivalent to the expected peak ground acceleration (g) on a site with rocky subsoil subjected to an earthquake with an average return period of 1 in 500 years. Z = 0.13 applies in Auckland (the country's most populated city) and Z = 0.39 applies in Hastings. Considering these relative hazards, the capacity/demand (C/D) ratio for four different assessment scenarios are summarised in Table 5 for the walls tested in this program.

Demands considered in Table 5 were based on the design basis earthquake (DBE) for the ultimate limit state (ULS) for each of the two cities in which the test buildings resided (i.e., Auckland and Hastings). The natural period of the representative system, T_p, was determined in accordance with Derakhshan et al. (2014). The average calculated natural periods for test walls with rigid top restraints (RC) and those that were propped (timber) were 0.40 and 0.85 seconds, respectively. With the highest calculated natural period for any single test wall being 0.94 seconds, all test walls had a part spectral shape coefficient, $C_i(T_p)$ between 1.6 and 2.0 (NZS 1170.5:2004). Assuming a shallow subsoil site class, non-ductile OOP behaviour of the wall (which is appropriate for a peak force-based assessment), a part risk factor, R_p = 1.0, a building importance level of 2 (representing a normal building, and hence, warranting the consideration of a DBE with an average return period of 1 in 500 years), and the full-height wall geometries and densities, the C/D ratio for each of the test walls was able to be determined for each of four scenarios as summarised in Table 5. Please note when considering these C/D ratios that an inherent conservativeness exists within both force-based assessments and one-way vertical flexural analyses.

Table 5: Summary of C/D ratios (considering effective full-height loaded capacities and assuming sites with shallow subsoils)

Test ID	Top edge restraint condition	Capacity/demand (C/D) ratios based on effective force-based capacity and NZS 1170.5:2004 "Parts and Components" demands							
		Auckland, ground floor of 4.5 m tall building		Auckland, third floor of 12 m tall building		Hastings, ground floor of 4.5 m tall building		Hastings, third floor of 12 m tall building	
		Demand (g)	C/D	Demand (g)	C/D	Demand (g)	C/D	Demand (g)	C/D
Auc-W1A	Propped	0.43	50%	0.95	23%	1.30	17%	2.85	8%
Auc-W1B	Propped	0.43	106%	0.95	48%	1.30	35%	2.85	16%
Auc-W2	Propped	0.35	67%	0.77	30%	1.05	22%	2.30	10%
Auc-W3	Propped	0.38	75%	0.83	34%	1.14	25%	2.50	11%
Has-W1	Rigid	0.46	284%	0.98	134%	1.38	95%	2.93	45%
Has-W2	Rigid	0.46	205%	0.98	96%	1.38	68%	2.93	32%
Has-W3	Propped	0.42	77%	0.89	36%	1.26	26%	2.68	12%
Has-W4	Propped	0.42	74%	0.89	35%	1.26	25%	2.67	12%
Has-W5	Rigid	0.46	174%	0.98	82%	1.38	58%	2.93	27%
Has-W6	Rigid	0.46	363%	0.98	171%	1.38	121%	2.93	57%

8. DETERMINATION OF EFFECTIVE SOLID-WALL THICKNESS FOR USE IN EXISTING ANALYTICAL MODELS

The experimentally measured force and displacement capacities for the six cavity walls tested in with propped top supports are summarised in Table 6. The experimental results were compared to the expected results from the procedure proposed by Derakhshan et al. (2014) by altering the values assumed for the effective solid wall thickness measured across the mortar joints until the model and experimental values aligned. This effective solid wall thicknesses, $b_{w,eff,exp}$, is also assumed to represent the wall thickness assumed to calculate wall weights in the Derakhshan et al. (2014) method. Note that the ratio $b_{w,eff,exp} / b_{wythes}$ shown in Table 6 is much higher for all test walls than the effective wall thickness recommended for OOP assessment by the British Standard (BS) (2005) which is only two-thirds of b_{wythes} (i.e., the sum of the two wythe thicknesses of the wall excluding the cavity). For the test walls listed in Table 6, the average crack height was $h_1 = 0.70(h_1 + h_2)$ which is nearly equal to but slightly higher than the crack height recommended by Derakhshan et al. (2014).

The experimentally measured force capacities for the four cavity walls tested with rigid top supports (and, hence, with arching action) are summarised in Table 7. The experimental results were compared to the expected results for strength capacity based on the procedure proposed by Angel et al. (1994) by altering the values assumed for the wall thickness measured across the mortar joints until the model and experimental values aligned. For the test walls listed in Table 7, the average crack height was $h_1 = 0.51(h_1 + h_2)$ which is nearly equal to the crack height assumed in the model by Angel et al. (1994).

Table 6: Summary of test results for walls tested without arching action and comparison to expected results

Test ID	Experimental test results			Derakhshan et al. (2014) model			Experimentally determined effective solid wall thickness, $b_{w,eff,exp}$	Model results / experimental results			
	Max effective* post-crack load, $F_{o,exp}$ (kN)	Instability displacement, $\Delta_{ins,exp}$ (mm)	Instability drift, $\Theta_{ins,exp}$ (mm)	Max post-crack load, F_o (kN)	Instability displacement, Δ_{ins} (mm)	Instability drift, Θ_{ins} (%)		F_o / $F_{o,exp}$	Δ_{ins} / $\Delta_{ins,exp}$	Θ_{ins} / $\Theta_{ins,exp}$	$b_{w,eff,exp}$ / b_{wythes}
Has-W3	5.7	258	10.3%	4.9	290	11.0%	290	0.87	1.12	1.07	1.29
Has-W4	5.4	249	9.9%	4.7	282	10.7%	282	0.86	1.13	1.08	1.25
Auc-W1A	2.9	-	-	2.9	217	10.8%	217	1.00	-	-	1.01
Auc-W1B	6.0	293	15.5%	5.7	307	15.2%	307*	0.95	1.05	0.98	1.43
Auc-W2	3.1	235	11.9%	3.2	228	11.3%	228	1.03	0.97	0.95	1.06
Auc-W3	3.7	246	14.2%	3.7	247	12.3%	247	0.99	1.00	0.86	1.15

*exceeds total out-to-out thickness of wall including cavity

Table 7: Summary of test results for walls tested with arching action and comparison to expected results for load-based capacity

Test ID	Experimental test results	Angel et al. (1994) model	Experimentally determined effective solid wall thickness, $b_{w,eff,exp}$	Model results / experimental results	
	Max effective* post-crack load, $F_{o,exp}$ (kN)	Max post-crack load, F_o (kN)		F_o / $F_{o,exp}$	$b_{w,eff,exp}$ / b_{wythes}
Has-W1	22.7	22.5	134	0.99	0.60
Has-W2	16.4	16.2	124	0.99	0.55
Has-W5	13.9	14.1	120	1.01	0.53
Has-W6	29.0	29.1	143	1.00	0.64

URM cavity walls with arching action are far more sensitive to the assumed effective wall thickness based on the parameters in the existing models. Walls with arching action have an expected OOP load capacity that is related to the slenderness ratio to a power of 4, whereas walls without arching action (and without any overburden load) have an expected OOP load capacity that is related to the slenderness ratio to a power of only 2 (see Figure 5 wherein the hypothetical walls were assumed to have the same properties as Has-W1, but the effective solid-wall thickness, $b_{w,eff}$,was incrementally adjusted). The estimated OOP load capacities of walls with partially rigid restraints can presumably be determined by utilising the adjustments factors for flexible frames in Angel et al. (1994), although further testing of this variable is desirable.

9. CONCLUSIONS AND RECOMMENDATIONS

Significant results that can be drawn from this research program are as follows:

- Top rigid restraint from the building frame causing “arching action” can greatly increase the OOP capacity of vertically-spanning infill walls. For relatively low slenderness ratios (i.e., ratios of wall height to effective thickness), the ratio of strength-based capacity between a cavity wall with arching action and a similarly retrofitted cavity wall without arching action (e.g., Has-W1 and Has-W3, respectively) was observed to be as high as 4;

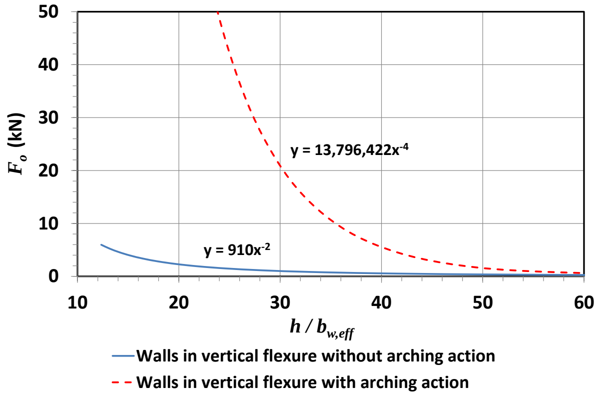


Figure 5: Sensitivity of two considered OOP wall capacity models to changes in slenderness ratio

- Cavity tie retrofits with adequate spacing, as well as adequate compressive and shear stiffness, can greatly improve the OOP capacity of URM cavity walls, especially for walls with arching action. For a force-based analysis in one case-study, the cavity wall OOP strength was more than doubled when comparing the existing condition (i.e., Has-W5) to the condition in which new ties were vertically spaced at approximately 150 mm (i.e., Has-W6);
- The adhesive cavity ties considered in this study were not notably more useful to improving OOP cavity wall performance than were more easily installed mechanical cavity ties;
- In accordance with the values shown in Table 5, most cavity infill walls with rigid supports and arching action are expected to perform well compared to design basis earthquake demands in Auckland. However, when subjected to higher spectral demands by the design basis earthquake in Hastings, especially at higher storeys, these walls may still be at risk of having their maximum strength exceeded. Walls without arching action (more similar to URM load-bearing walls) appear to be susceptible to strength exceedance when subjected to design basis earthquake demands in either city; and
- The OOP assessment methods proposed by Derakhshan et al. (2014) and Angel et al. (1994) as recommended for use in New Zealand (NZSEE 2006) are viable for assessing cavity walls in one-way vertical flexure provided the effective solid-wall thickness is determined firstly. Application of this observation to untested walls would presumably be contingent on the considered walls having geometric and material properties for both masonry and cavity ties similar to the specimen components considered in the reported experimental program.

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