COMPARISON BETWEEN PREDICTED URM WALL OUT-OF-PLANE STRENGTH-BASED CAPACITY AND IN SITU PROOF TEST RESULTS

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ABSTRACT

Unreinforced masonry (URM) building construction is prominent in the form of load-bearing, partition, and infill walls. Significant out-of-plane (OOP) failures of URM walls often occur during moderate and severe earthquake shaking and such walls are often identified in structural engineering assessments as being amongst the most vulnerable elements to OOP demands, especially earthquakes. For undamaged, in situ wall conditions where material properties are known and boundary conditions reflect idealised conditions assumed in analytical predictive models, these predictive models are easily applied, although the accuracy of the model outputs may still not be well understood. Furthermore, when in situ conditions do not reflect idealised conditions assumed in analytical predictive models, engineers are often uncertain as to which analytical models and inputs are most appropriately applied. Hence, an analytical campaign was undertaken to provide specific examples for structural engineering practitioners assessing the OOP seismic behaviour of URM walls, and the predictive results reported herein were compared to previously reported experimental results of eighteen tests on existing URM walls performed in situ. The considered wall configurations represented a variety of geometries, boundary conditions, pre-test damage states, and material properties. The average ratio and associated coefficient of variation (CV) of predicted strengths to measured strengths were determined to be 0.84 (CV 0.56) and 0.93 (CV 0.25) for the "unbounded" and "bounded" wall conditions, respectively, and corresponding recommendations for analytical assessment were made for practicing engineers.

KEYWORDS: *unreinforced masonry (URM), earthquakes, out-of-plane, infill walls, airbag proof-testing, analytical methods*

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INTRODUCTION

The earthquake vulnerability of buildings constructed using conventional British architecture with unreinforced fired clay brick masonry (URM) prior to the introduction of modern earthquake loading standards is especially well-known in Australasia [1]. Nonetheless, few such existing URM structures have been retrofitted to resist design basis earthquake (DBE) forces, despite the prominence of this construction in the form of load-bearing, partition, and infill walls. In particular, significant out-of-plane (OOP) failures of URM walls often occur during moderate and severe earthquake shaking. Predictive analytical models that apply to particular wall configurations have been developed over the past few decades. However, the accuracy of such methods relative to in situ proof testing results has not been widely reported. Furthermore, these predictive methods often involve the assumption of idealised boundary conditions and pre-existing damage states that may not exist in "real world" configurations. Hence, a study that compares the accuracy of widely used predictive methods and assumed input values to the results of experimental in situ tests is lacking and was the subject of the investigation reported herein. The experimental results considered are derived from the testing program carried out and previously reported by Walsh et al. [2], including eighteen in situ URM walls wherein lateral forces were applied using airbags to simulate distributed OOP demands. The referenced test set of URM walls represented a variety of geometries, boundary conditions, pre-test damage states, and material properties, such that the compared predictive results reported herein may be especially useful for structural engineering practitioners.

PREDICTIVE MODELS FOR THE OOP CAPACITY OF URM WALLS

The predictive analytical methods considered herein (and the in situ wall conditions to which they are applied) can generally be categorised into "unbounded" and "bounded" wall conditions. In the case of unbounded URM walls (such as those generally found in buildings where URM is a load-bearing element or a continuous façade/parapet feature), practicing engineers may consider referencing the assessment methodology of the Australian Standard (AS) for masonry design [3] and related supplemental references [4]. The AS [3] design method utilises a virtual work-based, one-way or two-way flexural analysis including weighted components of vertical flexure, horizontal flexure, and diagonal flexure where applicable, while accounting for different material properties and boundary conditions. Griffith and Vaculik [5] validated the relative accuracy of the AS [3] design method with experimental laboratory testing results, provided that return walls were assumed to provide only partial moment restraint such that the vertical edge restraint factor, R_f, equalled 0.5.

Load-bearing URM walls typically have timber diaphragms [3, 6] which have been experimentally shown to effectuate little to no compressive strut "arching" action in URM walls under OOP loading [7, 8]. In contrast, URM infill walls which are bounded by relatively rigid elements, such as RC frames, may form compressive strut "arching" mechanisms while deforming OOP, which generally increases the OOP strength of the URM walls as compared to unbounded wall conditions. Flanagan and Bennett [9] compared the accuracy of the empirically based Dawe and

Seah [10] predictive model for estimating the OOP strength of URM infill walls and the analytically derived predictive model proposed by Abrams et al. [11]. Flanagan and Bennett [9] concluded that the Dawe and Seah [10] model produced slightly more accurate predictions for the majority of a large experimental laboratory data set. Furthermore, the Dawe and Seah [10] predictive model accounts for URM infill bounding restraints resulting in either one-way or two-way flexure. In comparison, the Abrams et al. [11] predictive model only accounts for one-way flexure in the stronger of two directions, where applicable [9]. However, Abrams et al. [11] did uniquely provide OOP strength reduction factors to account for in-plane damage preceding OOP loading on URM infill walls. Hence, for the purposes of the analytical study reported herein, OOP strength for bounded walls was predicted using the revised Dawe and Seah [10] model as proposed by Flanagan and Bennett [9] and as recommended for use by the Masonry Standards Joint Committee (MSJC) [12]. Strength reduction factors to account for in-plane damage recommended by Abrams et al. [11] were incorporated where appropriate.

Table 1: Summary of recommended predictive OOP performance models and associated applications

<table>
<tr><th>Predictive model</th><th>Applicable performance metric</th><th>Top and bottom edge restraints</th><th>Side edge restraints</th><th>Model assumptions and applications</th></tr>
<tr><td>AS [3]</td><td>Strength</td><td>Timber diaphragm or URM wall on contiguous levels*</td><td>URM return walls / piers</td><td>• one-way or two-way flexure
• can accommodate edge restraints with varying flexural rigidity
• can accommodate overburden loads</td></tr>
<tr><td>Derakhshan et al. [13]</td><td>Displacement at peak strength</td><td>Timber diaphragm or URM wall on contiguous levels*</td><td>Free (unrestrained)**</td><td>• one-way vertical flexure only
• simply supported restraints top and bottom
• can accommodate overburden loads</td></tr>
<tr><td>NZSEE [6]</td><td>Instability displacement</td><td>Timber diaphragm or URM wall on contiguous levels*</td><td>Free (unrestrained)**</td><td>• one-way vertical flexure only
• simply supported restraints top and bottom
• can accommodate overburden loads</td></tr>
<tr><td>Flanagan and Bennett [9]</td><td>Strength</td><td>RC slab or RC beam***</td><td>RC columns***</td><td>• one-way or two-way flexure / arching (rigid elements – RC or steel – must be present on at least two opposing sides)
• formation of compressive strut “arching” mechanisms
• can accommodate bounding frame restraints with varying flexural rigidity</td></tr>
<tr><td>Abrams et al. [11]</td><td>Strength reduction</td><td colspan="2">RC slab or RC frame element*** (stronger of two arching directions – horizontal or vertical – assumed to govern)</td><td>• one-way flexure / arching only
• utilised in the study reported herein for URM infill OOP strength reduction due to preceding in-plane damage</td></tr>
<tr><td>Flanagan and Bennett [9]</td><td>Drift at peak strength</td><td>RC slab or RC beam***</td><td>n/a</td><td>• one-way vertical flexure / arching only</td></tr>
<tr><td colspan="5">*RC bond beams are also present at floor levels in many pre-war buildings with load-bearing URM walls and timber diaphragms in Australasia, but without vertical rigid elements (i.e., RC columns) to restrain the RC bond beams against vertical deflection, the RC bond beams are generally not assumed to effectuate compressive strut “arching” mechanisms in the URM walls under OOP loading.
** Recommended for best-practice use in the case of isolated piers between window/door openings or for long parapets (with no overburden loads), such that one-way vertical flexure (i.e., horizontal cracking rather than vertical or diagonal cracking) is likely to govern OOP collapse.
***URM infill walls may also be bounded by steel framing, but such an arrangement is far less common in Australasia.</td></tr>
</table>

While not considered in the study reported herein, other analytical models exist for predicting the drift/displacement performance of walls under OOP loads. For prudence, these displacement-based models [6, 9, 13] are listed in Table 1 in addition to the predictive models currently considered in the study reported herein for estimating OOP strength. Various alternative methods for predicting the OOP behaviour of URM walls have been reported elsewhere [14].

TEST WALL CONDITIONS

Test walls were located within six different buildings in New Zealand: the Weir House (WH) estate in Wellington (constructed 1932), the Oriental Bay (WO) apartments in Wellington (early 1900s), the Wellington Railway Station (WR, 1937), an automotive garage (AG) in the Auckland CBD (1958), a retail building (AO) located in Orakei, Auckland (1938), and a mixed-used building on Kingston Street (AK) located in the Auckland CBD (1927). Geometries and pre-test damage states are summarised in Table 2. Additional information about the existing wall conditions and how lateral forces were applied to the test walls using airbags is provided by Walsh et al. [2].

Table 2: Summary of test wall geometries, boundary conditions, and preparations

Test ID	Length (mm)	Full in situ height* (mm)	Brick thickness (mm)	Top edge restraint	Side (vertical) edge restraints	Bottom edge restraint	Features and preparations
WH1	4100	3600	95	RC slab with contiguous URM wall above	700x500 mm RC column and URM return wall	RC slab with contiguous URM wall below	Plaster 15–20 mm thick each side
WH2	3850	2730	95	Gypsum board (free)	RC column and URM return wall	RC slab	Plaster 15–20 mm thick each side, existing minor cracks
WH3	3480	2730	95	Gypsum board (free)	RC shear wall and timber wardrobe	RC slab	Plaster 15–20 mm thick each side, existing minor cracks
WO1A	3900	2740	110	Timber (lateral only)	URM return walls both sides	URM / RC	Plaster 15–20 mm thick each side
WO1B	3900	2740	110	Timber (lateral only)	URM return walls both sides	URM / RC	Plaster 15–20 mm thick each side, horizontal 50 mm deep cut
WO1C	3900	2740	110	Timber (lateral only)	URM return walls both sides	URM / RC	Plaster 15–20 mm thick each side, horizontal and vertical 50
WO2	2600	2740	110	Timber (overburden)	URM return wall and short door opening	URM / RC	Plaster 15–20 mm thick each side, load-bearing wall (overburden load)
WR1	2180	4280	108	127 mm RC slab	Free (unrestr.) both sides	RC slab on grade	Side edges saw cut free
WR2A	2662	4342	108	127 mm RC slab	URM return wall and URM pier	RC slab on grade	WR2A tested in its existing condition prior to saw cutting
WR2B	1915	4342	108	127 mm RC slab	Free (unrestr.) both sides	RC slab on grade	Side edges of WR2B saw cut free after testing WR2A

Table 2: Summary of test wall geometries, boundary conditions, and preparations

Test ID	Length (mm)	Full in situ height* (mm)	Brick thickness (mm)	Top edge restraint	Side (vertical) edge restraints	Bottom edge restraint	Features and preparations
WR3	3385	3100	108	127 mm RC slab	Free (unrestr.) both sides	127 mm RC slab	Side edges saw cut free (one side was saw cut above existing door opening)
WR4	1900	3100	108	127 mm RC slab	Free (unrestr.) and tall door	127 mm RC slab	One side edge saw cut free and other side edge had nearly full
WR5	2580	2980	108	Timber (lateral only)	URM return wall and short door opening	RC slab	
WR6	1305	2980	108	Timber (lateral only)	Free (unrestr.) both sides	RC slab	Side edges saw cut free
AG1	4400	3400	112.5	280x150mm RC beam	305x265mm concrete-encased steel columns both sides, contiguous infill on one side	RC slab on grade	Brick masonry veneer (as part of cavity infill wall) removed prior to testing
AG2	4400	3400	112.5	280x150mm RC beam	305x265mm concrete-encased steel columns both sides, contiguous infill	RC slab on grade	Brick masonry veneer (as part of cavity infill wall) removed prior to testing, simulated in-plane cracking with 50 mm deep cut in X-shape across entire panel
AO1	3380	2655	109	300x375mm RC beam	350x350mm RC column (interior) with contiguous infill and 300x300mm RC column (exterior)	Timber	
AK1	This wall is not considered in this analysis because it was tested with in situ cavity ties that are outside the scope of the considered predictive methods. See Walsh et al. [2] for further information.						
AK2	1450	2750	75	300x475mm RC beam	Free (unrestr.) both sides	300x475mm RC beam	Vertically cut through the 75 mm brick and removed original cavity steel wire ties
*Refer to Walsh et al. [2] for the distinction between full in situ height and test height							

MATERIAL PROPERTIES AND GEOMETRIES

Brick, mortar, and masonry prism samples were extracted from the test walls and tested in accordance with the relevant ASTM standards (see Walsh et al. [2] for the complete list of standards). The gross cross-section of bricks was assumed for determining all material strengths. A summary of the material test results is included in Table 3 where all strength values are in units of MPa, unless noted otherwise. When it was not possible to test for certain material strengths, empirical equations were used to estimate the predicted mean values, as described by Walsh et al. [2].

Table 3: Summary of measured and estimated masonry material characteristics (all strength values in MPa unless noted otherwise)

Test wall(s)	Parameter	Brick compression strength, f'_b	Mortar compression strength, f'_j	Masonry prism compression strength, f'_m	Masonry prism bond rupture strength, f'_{fb}	Brick rupture strength (modulus of rupture), f'_{mr}	Masonry prism density, ρ_m (kg/m^3)
WH1 WH2 WH3	Mean	12.5	26.7	13.8	0.80	1.5	1650
	CV	Est.	Est.	0.50	0.26	Est.	Est. (hollow)
	#			4	3		
WO1 WO2	Mean	25.6	12.6	18.7	0.38	3.1	1807
	CV	0.28	0.29	Est.	Est.	Est.	Est.
	#	7	18				
WR1	Mean	24.6	9.9	16.9	0.30	3.0	1780
	CV	0.15	0.30	Est.	Est.	Est.	Est.
	#	4	6				
WR2	Mean	42.0	11.2	26.2	0.34	5.0	1878
	CV	0.09	0.23	Est.	Est.	Est.	Est.
	#	4	6				
WR3 WR4	Mean	33.0	7.9	19.5	0.24	4.0	1806
	CV	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.
	#						
WR5	Mean	37.4	8.0	21.6	0.24	4.5	1829
	CV	0.12	0.31	Est.	Est.	Est.	Est.
	#	3	6				
WR6	Mean	28.5	7.8	17.5	0.23	3.4	1783
	CV	0.25	0.26	Est.	Est.	Est.	Est.
	#	3	5				
AG1 AG2	Mean	35.5	13.9	9.4	0.42	3.6	1720
	CV	0.08	0.09	0.30	Est.	0.23	0.03
	#	5	5	2		4	3
AO1	Mean	27.6	8.4	17.5	0.25	3.3	1783
	CV	0.29	0.41	Est.	Est.	Est.	Est.
	#	4	6				
AK1 AK2	Mean	8.0	1.2	3.8	0.04	1.0	1628
	CV	0.27	0.22	Est.	Est.	Est.	Est.
	#	7	13				

Notes: mean = average of measured values; CV = coefficient of variation defined as the sample standard deviation divided by the mean; # = number of test samples; Est. = estimated (predicted mean) value by empirical equation (refer to Walsh et al. [2]); Avg. = average of corresponding WR5 and WR6 values

The expected concrete compression strength for each relevant building was determined as follows: 26 MPa for the Wellington Railway Station (WR) per Peng and McKenzie [15]; 42 MPa for the Auckland Victoria Street automotive garage (AG) estimated as the specified strength for contemporary concrete of 21 MPa [16] multiplied by 2.0 to account for age and overstrength [17]; 34 MPa for the Auckland Orakei retail building (AO) estimated as the specified strength for contemporary concrete of 17 MPa [16] multiplied by 2.0 to account for age and overstrength [17], and 28 MPa for both the Wellington Weir House (WH) and the Auckland Kingston Street (AK) building, estimated as the specified strength for contemporary concrete of 14 MPa [16] multiplied by 2.0 to account for age and overstrength [17]. The elastic modulus of concrete, E (MPa), was estimated as a function of the compressive strength of concrete, f'_{co} (MPa), assuming $E = 3320\sqrt{f'_{co}} + 6900$ in accordance with NZS [18].

COMPARISON OF MEASURED AND PREDICTED WALL CAPACITIES

A summary of ratios of predicted and measured performance and assumptions made as part of the predictive modelling inputs are listed in Table 4. Note that most of the test walls were assessed as being either unbounded (i.e., having no arching action per AS [3]) or being bounded (i.e., having arching action per Flanagan and Bennett [9]) and compared explicitly to the appropriate predictive model. However, three test walls (WH1, WR2A, and AO1) were reasonably deemed as appropriate to be assessed with either predictive model due to having potentially rigid bounding elements in one flexural direction but not in the other. For walls assessed for strength using the AS [3] model, all but one wall were assumed to experience two-way flexure during OOP deformation. The exception was test wall WR6 which was assessed using AS [3] criteria assuming only one-way vertical flexure. As noted in Table 4, design length, L_d , and design height, H_d , values were assumed either equal to full or half of the in situ wall dimensions depending on the presence of boundary restraints in the respective directions. Side (vertical) edge rotational restraints factors, R_{f1} and R_{f2} , were assumed equal to 0.0, 0.5, or 1.0 for restraints consisting of timber, URM [5], and RC elements respectively.

For walls assessed for strength using the Flanagan and Bennett [9] model, the relative stiffness factors for the bounding elements, α and β, were determined in accordance with the recommendations of MSJC [12] whereby the average values used in the model, α_{avg} and β_{avg}, as noted in Table 4 were determined by averaging the respective factors for the elements on opposite edges from each other. RC slabs on grade were assumed to provide the maximum stiffness value permissible in the model of 50.0. Bounding elements separating contiguous URM infill panels were also assumed to provide the maximum stiffness value permissible in the model of 50.0 (e.g., the column separating test walls AG1 and AG2). Elements unlikely to provide enough relative stiffness to effectuate significant arching action (e.g., timber framing) were assigned relative stiffness factors of 0.0. Other elements (e.g., RC slabs and beams) were assigned relative stiffness factors proportional to their flexural rigidities (*EI*). Where top or bottom bounding elements were RC slabs, the moment of interia, I , was calculated assuming an effective flange width of 16 times the thickness of the slab per the recommendation of NZS [18]. RC sections were assumed to be uncracked. Strength reduction due to simulated in-plane damage was assumed in accordance with the recommendations of Abrams et al. [11]. Even though the strength reduction factors proposed by Abrams et al. [11] were based on tests of masonry infill panels with arching action and, hence, most appropriately applied to test wall AG2, assumed strength reduction factors were also applied to the predicted OOP strengths of test walls WO1B and WO1C due to a lack of existing, relevant research for pre-damaged URM walls without arching action.

The average ratio and associated coefficient of variation (CV) of predicted strengths to measured strengths were determined to be 0.84 (CV 0.56) and 0.93 (CV 0.25) for the AS [3] and Flanagan and Bennett [9] methods, respectively. Note that all cases of predicted strength ratios higher than 1.0 listed in Table 4 were associated with test walls that may not have been loaded to their peak force capacities [2]. Test walls WR5 and WR6 had notably low predicted strength ratios of 0.24 and 0.25, respectively. In the case of test wall WR5, neglecting the contribution from the spandrel above the door opening to side (vertical) edge rotational restraint may have contributed to the significant underestimation of OOP strength. Note however, that the contribution from the spandrel above the door opening was also neglected in predicting the OOP strength of test wall WO2. In the case of test wall WR6, the vertical saw cut preparations were executed in such a fashion that a relatively deep spandrel remained above the portion of the wall tested in one-way vertical flexure. This spandrel may have applied a greater rotational restraint condition and/or overburden load to the wall during OOP deformation than was assumed, thus increasing the test wall's OOP strength and reducing the accuracy of the one-way flexural model applied to it per AS [3]. If the ratios for test walls WR5 and WR6 were neglected, the average ratio and associated CV of predicted to measured strengths listed in Table 4 would become 0.97 (CV 0.42) for the AS [3] method.

As shown at the end of Table 4, predicted capacity reduction factors of 0.55 and 0.70 would ensure 100% of the test wall specimens considered herein were underpredicted for strength capacity using the AS [3] and Flanagan and Bennett [9] methods, respectively, notwithstanding that some test walls were not able to be experimentally tested to their ultimate capacities [2]. By comparison, both AS [3] and MSJC [12] specify the use of a capacity reduction factor of 0.60 for flexure in unreinforced masonry. Note that AS [3] and MSJC [12] reduction factors are used in conjunction with other factors of safety inherent to new design practice (e.g., specified lower-bound material strength) that were not considered in the predicted capacities of existing walls as reported herein.

CONCLUSIONS

On average, both predictive strength models produced relatively accurate results using the assumed inputs as noted in Table 4 (for various boundary restraint conditions and pre-damage states). The variance of the results was notably high for the AS [3] predictive model, which may be the result of the AS [3] model accommodating a larger range of configurations and boundary condition types compared to the Flanagan and Bennett [9] method. The predictive strength models produced similar results for two of the three walls (WH1 and AO1) assessed using both "unbounded" and "bounded" predictive models. In contrast, wall WR2A was predicted to have varying performances using the two strength models, perhaps due to its comparably unique boundary conditions. The mean ratio of predicted to experimental strength capacity using the Flanagan and Bennett [9] method of 0.93 contradicts the conclusion drawn by Flanagan and Bennett that the Dawe and Seah [10] method systematically overpredicts the capacity of walls by a factor of 1.09, and for which Flanagan and Bennett proportionally adjusted the coefficient of the Dawe and Seah predictive equation. Note, however, that Flanagan and Bennett [9] referenced laboratory tests as opposed to the in situ experimental tests considered herein. Predicted capacity reduction factors of 0.55 and 0.70 would ensure 100% of the test wall specimens considered herein were underpredicted for strength capacity using the AS [3] and Flanagan and Bennett [9] methods, respectively.

Table 4: Comparison of predicted and measured wall performance

Test ID	Experimental equivalent force-based capacity (kN)	Experimental equivalent force-based capacity (g)	Ratio of predicted to experimental force-based capacity		Assumptions used in the predictive model
			Strength [3]	Strength [9]	
WH1	72.7	3.20	0.82	0.84	AS [3]: h_u = 160 mm; l_u = 300 mm; t_j = 15 mm; L_d = 2050 mm; H_d = 1800 mm; R_{f1} = 1.0 (RC column); R_{f2} = 0.5 (URM return wall) Flanagan and Bennett [9]: α_{left} = 50 (RC column); α_{right} = 0 (assumed URM return wall would not effectuate arching action on its own, but in conjunction with RC column on other side, would effectuate some arching action, in contrast to timber); α_{avg} = 25.0 $\beta_{top} = \beta_{bottom} = \beta_{avg}$ = 50 (RC slab above and below with contiguous infill)
WH2	46.2	2.86	0.75	-	h_u = 160 mm; l_u = 300 mm; t_j = 15 mm; L_d = 1925 mm; H_d = 2730 mm; R_{f1} = 1.0 (RC column); R_{f2} = 0.5 (URM return wall)
WH3	46.8	3.20	0.69	-	h_u = 160 mm; l_u = 300 mm; t_j = 15 mm; L_d = 1740 mm; H_d = 2730 mm; R_{f1} = 1.0 (RC column); R_{f2} = 0.0 (timber wardrobe)
WO1A	23.6	1.13	1.81	-	h_u = 76 mm; l_u = 230 mm; t_j = 18 mm; L_d = 1950 mm; H_d = 1370 mm; $R_{f1} = R_{f2}$ = 0.50 (URM return walls)
WO1B	39.7	1.91	0.92	-	h_u = 76 mm; l_u = 230 mm; t_j = 18 mm; L_d = 1950 mm; H_d = 1370 mm; $R_{f1} = R_{f2}$ = 0.50 (URM return walls) Strength reduction factor to account for in-plane damage, R_I = 0.85 (Abrams et al. [11])
WO1C	47.8	2.30	0.67	-	h_u = 76 mm; l_u = 230 mm; t_j = 18 mm; L_d = 1950 mm; H_d = 1370 mm; $R_{f1} = R_{f2}$ = 0.50 (URM return walls) Strength reduction factor to account for in-plane damage, R_I = 0.75 (Abrams et al. [11])
WO2	22.9	1.65	0.84	-	h_u = 76 mm; l_u = 230 mm; t_j = 18 mm; L_d = 2600 mm; H_d = 1370 mm; R_{f1} = 0.5 (URM return wall); R_{f2} = n/a (door opening)
WR1	24.5	1.39	-	0.97	$\alpha_{left} = \alpha_{right} = \alpha_{avg}$ = 0.0 (saw cut free edges) β_{top} = 36.3 (RC slab with effective width = 16 × thickness per NZS 2006); β_{bottom} = 50.0 (RC slab on grade); β_{avg} = 43.2
WR2A	41.6	1.81	1.52	0.91	AS [3]: h_u = 78 mm; l_u = 223 mm; t_j = 13.5 mm; L_d = 1331 mm; H_d = 2171 mm; $R_{f1} = R_{f2}$ = 0.50 (URM return wall or pier) Flanagan and Bennett [9]: $\alpha_{left} = \alpha_{right} = \alpha_{avg}$ = 0.0 (URM return wall and UMR pier) β_{top} = 32.9 (RC slab with effective width = 16 × thickness per NZS [18]); β_{bottom} = 50.0 (RC slab on grade); β_{avg} = 41.4
WR2B	20.7	1.25	-	1.41	$\alpha_{left} = \alpha_{right} = \alpha_{avg}$ = 0.0 (saw cut free edges) β_{top} = 38.7 (RC slab with effective width = 16 × thickness per NZS [18]); β_{bottom} = 50.0 (RC slab on grade); β_{avg} = 44.4
WR3	43.6	2.17	-	1.04	$\alpha_{left} = \alpha_{right} = \alpha_{avg}$ = 0.0 (saw cut free edges) $\beta_{top} = \beta_{bottom} = \beta_{avg}$ = 29.1 (RC slab with effective width = 16 × thickness per NZS [18])
WR4	38.6	3.43	-	0.88	$\alpha_{left} = \alpha_{right} = \alpha_{avg}$ = 0.0 (saw cut free edges) $\beta_{top} = \beta_{bottom} = \beta_{avg}$ = 38.9 (RC slab with effective width = 16 × thickness per NZS [18])

Table 4: Comparison of predicted and measured wall performance

Test ID	Experimental equivalent force-based capacity (kN)	Experimental equivalent force-based capacity (g)	Ratio of predicted to experimental force-based capacity		Assumptions used in the predictive model
			Strength [3]	**Strength [9]**	
WR5	65.1	4.37	0.24	-	h_u = 78 mm; l_u = 223 mm; t_j = 13.5 mm; L_d = 2580 mm; H_d = 1490 mm; R_{f1} = 0.5 (URM return wall); R_{f2} = n/a (door opening)
WR6	7.0	0.96	0.25	-	Only one-way vertical flexure considered (AS [3]; Think Brick [4])
AG1	61.3	2.16	-	1.00	α_{left} = 32.8 (column against door opening); α_{right} = 50.0 (column against contiguous infill); α_{avg} = 41.4 β_{top} = 18.4 (shallow RC beam); β_{bottom} = 50.0 (RC slab on grade); β_{avg} = 34.2
AG2	38.4	1.35	-	0.96	α_{left} = 50.0 (column against contiguous infill); α_{right} = 32.8 (column against door opening); α_{avg} = 41.4 β_{top} = 18.4 (shallow RC beam); β_{bottom} = 50.0 (RC slab on grade); β_{avg} = 34.2 Strength reduction factor to account for in-plane damage, R_I = 0.60 (Abrams et al. [11])
AO1	63.9	3.74	0.72	0.70	AS [3]: h_u = 76 mm; l_u = 225.5 mm; t_j = 13.5 mm; L_d = 1690 mm; H_d = 1328 mm; R_{f1} = R_{f2} = 1.0 (RC columns) Flanagan and Bennett [9]: α_{left} = 50 (larger interior column with contiguous infill on the opposite side); α_{right} = 39.8 (smaller exterior column); α_{avg} = 44.9; β_{avg} = 0.0 (assumed suspended timber floor not stiff enough to effectuate arching action)
AK1	19.9	1.81	This wall is not considered in this analysis because it was tested with in situ cavity ties that are outside the scope of the considered predictive methods.		
AK2	10.2	2.68	-	0.55	α_{left} = α_{right} = α_{avg} = 0.0 (saw cut free edges) β_{top} = β_{bottom} = β_{avg} = 50.0 (deep RC beam)
Avg.			**0.84**	**0.93**	-
Underprediction rate using predicted capacity reduction factor of 1.0			**82%**	**70%**	-
Underprediction rate using predicted capacity reduction factor of 0.75			**82%**	**90%**	-
Underprediction rate using predicted capacity reduction factor of 0.65			**91%**	**100%**	-
Underprediction rate using predicted capacity reduction factor of 0.55			**100%**	**100%**	-

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