PSEUDO-STATIC IN-PLANE TESTING OF TYPICAL NEW ZEALAND UNREINFORCED MASONRY WALLS

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ABSTRACT:

No new unreinforced masonry (URM) buildings have been constructed in New Zealand since the introduction of NZS 1900 in 1965. This code prohibited the use of URM as a building material due to its poor resistance to seismic forces. Nevertheless, many URM buildings still exist in New Zealand. Many of these form a significant part of the country's architectural heritage, and in order to reduce their associated seismic risk, a favourable option is to improve their seismic performance, rather than demolish the building. In order to ensure acceptable performance during an earthquake, seismic retrofit solutions must be applied to the building. It is first necessary to understand the existing performance of the building before appropriate retrofit solutions are implemented. A number of assessment methods are already used in New Zealand and are based on findings from overseas research. In-plane pseudo-static testing has been conducted on a single unreinforced masonry wall, which was built to replicate typical New Zealand construction of the early 20th Century. The results of this test are compared with the predicted performance calculated using existing New Zealand desktop assessment methods for unreinforced masonry. It is concluded that currently available guidelines successfully predicted the failure mode and limiting strength of a single URM wall with low axial load.

1 INTRODUCTION

New Zealand's unreinforced masonry (URM) building stock was constructed between about 1880 and 1950 (Stacpoole and Beaven 1972). Due to its poor resistance to seismic forces, the use of URM as a building material was explicitly outlawed in 1965 in most areas of New Zealand with the introduction of NZS 1900 (New Zealand Standards Institute 1965). Many URM buildings still exist in New Zealand. Of these, many are protected by the Historic Places Trust (Robinson and Bowman 2000), or if not actually protected (scheduled) are nevertheless an important part of New Zealand's architectural heritage. Legislation has recently been introduced in New Zealand where earthquake risk buildings must be improved to meet a required standard, or else removed (New Zealand Society for Earthquake Engineering 2006; Department of Building and Housing - Te Tari Kaupapa Whare 2007). Within this legislative framework the option of demolition may be more attractive to the building owner when compared to the investment associated with seismic retrofit of the structure. As many URM buildings form part of the country's heritage architecture, demolition in order to mitigate their seismic hazard is an unfavourable option. Thus for retrofit solutions to be viable, they must be cost-effective, and to facilitate this, accurate assessment of the structure's expected seismic behaviour takes on a greater importance.

A number of researchers have conducted tests on isolated URM walls (Magenes and Calvi 1992;

Manzouri et al. 1996; Magenes and Calvi 1997). It is generally accepted that walls will fail in the weakest mode of: sliding shear, rocking, toe compression or through diagonal tension cracking. Guidelines have been developed in New Zealand (New Zealand Society for Earthquake Engineering 2006) to determine which of these modes of failure will govern for a wall with particular material properties, axial load and boundary conditions. The purpose of the current research is to compare the predicted performance of URM walls constructed with New Zealand materials against results obtained through experimental testing, and to correlate experimental findings with those from researchers using materials native to countries other than New Zealand.

Researchers have also considered the behaviour of URM as part of a system (Abrams and Costley 1996; Paquette and Bruneau 1999; Abrams 2000; Abrams 2001; Paquette and Bruneau 2003). System level response considers the performance of an overall building, in contrast to component level response which in this instance considers isolated walls. It is particularly important to consider the performance of a system, and how components influence the overall system response. The results presented in this paper are of component URM elements. Further research will be conducted to investigate how these components influence the system level response of URM structures.

2 BACKGROUND

2.1 *New Zealand earthquake hazard*

New Zealand is located in a seismically active region and lies on the boundary of the Australian and Pacific tectonic plates. To the east of the North Island the Pacific plate is forced under the Australian plate. Under the South Island the two plates push past each other sideways, and to the south of New Zealand the Australian plate is forced under the Pacific plate (Figure 1).

Figure 1: The plate boundary in New Zealand (reproduced from (Institute of Geological and Nuclear Sciences 2005) with permission)

It is estimated that New Zealand has around 14,000 earthquakes each year; most are small, but between 100 and 150 have a magnitude sufficient to be felt. In the past 150 years, New Zealand has had around 15 earthquakes registering over M7.0 magnitude on the Richter scale, with a centre less than 30 km deep (Institute of Geological and Nuclear Sciences 2005).

3 MOTIVATION FOR RESEARCH

As part of a large-scale research project focused on developing retrofit solutions for New Zealand's earthquake risk buildings it has become evident that the accurate assessment of URM buildings is a high priority. It is necessary to assess the performance of an existing structure before a decision to apply any retrofit solutions can be made.

The laboratory test reported in this paper is the first in a series of tests aimed at providing a comprehensive picture of the seismic performance of URM structures in New Zealand. An understanding of the expected performance in terms of material properties, component level response and system level response will be gained. This will provide a platform against which to compare the building performance of retrofitted URM structures, and facilitate the implementation of cost-effective and economically viable retrofit solutions. Also, the influence of components on the overall building response will be investigated.

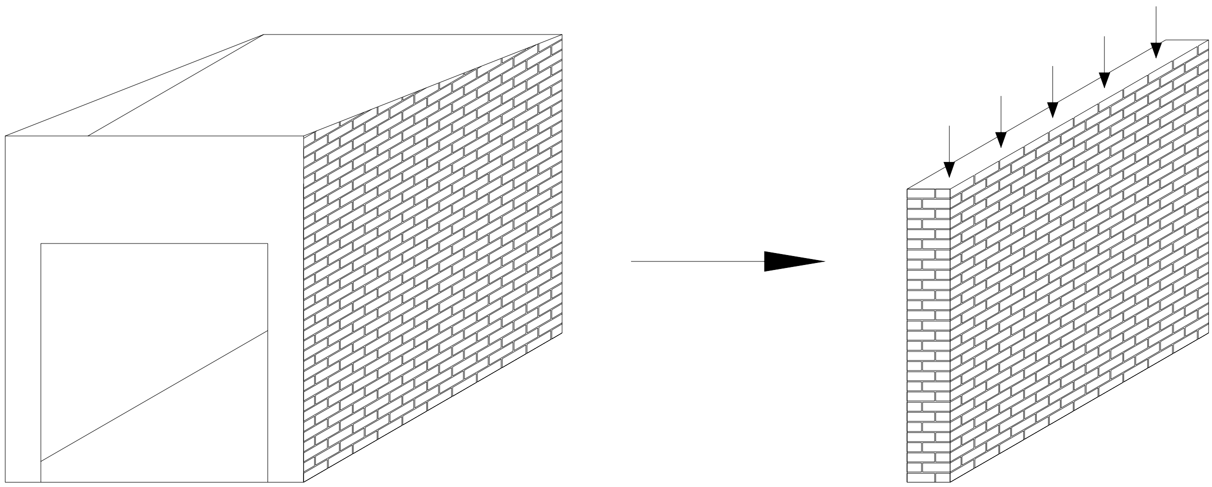
Specifically this test is aimed at investigating the post-elastic, non-linear response of New Zealand URM walls replicated from existing structures using old bricks and a mortar designed to mimic old construction techniques.

3.1 *Test objectives*

Previous published experimental research that considered existing New Zealand URM wall response has focused on out-of-plane behaviour (Blaikie 1999; Blaikie 2002). Guidelines on the assessment of in-plane wall response (New Zealand Society for Earthquake Engineering 2006) have mainly been derived from results obtained overseas (Magenes and Calvi 1992; Gambarotta and Lagomarsino 1997; Magenes and Calvi 1997; Magenes and Della-Fontana 1998; Kitching 1999; Foss 2001). The aim of the test reported here was to acquire experimental results from a URM in-plane wall test, against which the predicted response can be compared.

4 **TEST STRUCTURE**

A single three-leaf (wythe) wall was constructed using recycled bricks. The wall simulated a side wall in a single storey isolated URM structure, as shown in Figure 2.



(a) prototype building (b) test wall

Figure 2: Location of wall in a single storey isolated URM structure on which the test wall was simulated

The test wall measured 2500 mm long × 2500 mm tall × 360 mm thick (three leaves). Three-leaf walls are considered the most commonly occurring in New Zealand at the bottom storey of URM structures, and the bond pattern employed was Common (American) bond, with headers every 4th course. Common bond is also considered the most prevalent bond pattern in New Zealand. It was intentionally decided to construct the wall in a way that replicated the observed, often deteriorated, finish quality of walls in real buildings. A small axial load of 6 kN (0.01 MPa) was applied to the top of the wall, to simulate the axial load experienced on the side wall of a characteristic New Zealand single-storey isolated URM building, as shown in Figure 2. On such a building, this axial load is made up of the weight of the front parapet and a small roof load. A wall in an actual building as shown in Figure 2 includes the sloping parapet on top. The test wall had a level top surface, and the weight of the sloping parapet and roof load was approximated as a uniformly distributed load along the surface.

4.1 *Materials*

The wall was constructed using recycled bricks from a demolished building. The old mortar was removed and the surfaces of the bricks were prepared for new mortar before being reused in the test wall. The bricks were estimated to be between 60 and 100 years old. These bricks were used because the manufacturing processes for making new bricks are sufficiently different to substantially alter the properties and characteristics of the bricks. In particular the difference in porosity between currently manufactured bricks and old bricks means that the bond at the brick-mortar interface is much weaker in new bricks, using the mortar required for this test. It is believed that within a building there is significant variability in the bricks, and that reusing these bricks also introduced material variability into the test. Because this variability is natural and realistic, it was considered acceptable. There are boutique factories in New Zealand which still produce bricks according to the same processes employed to manufacture bricks when URM buildings were originally being constructed in New Zealand. Financial limitations precluded the use of these bricks to construct the test wall. Random samples of bricks were taken during construction of the wall and tested in compression. Prisms were also built during construction of the wall using randomly selected bricks, and tested in compression and flexural tension. The results of these tests are shown in Table 1. A weak mortar mix, ASTM type 'O' (1:2:9 cement/lime/sand by volume) was selected to simulate decayed mortar in old URM structures. Standard Portland cement, hydrated lime (Calcium Hydroxide) and river sand (gradation curve shown in Figure 3) were used in the mortar. Portland cement was widely available at the time when URM was a common building material in New Zealand in the early part of the 20th Century, as was hydrated lime. Results of tests conducted on mortar samples are summarized in Table 1. All material tests were conducted according to ASTM standards. A high coefficient of variation for each property indicates a high variability in the materials used, but this is considered acceptable because of the real variation in material properties in existing URM structures.

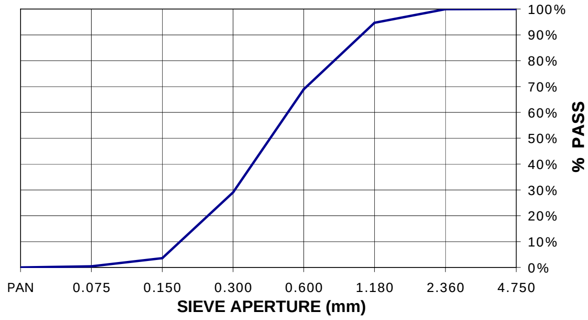


Figure 3: Mortar sand gradation curve

Table 1: Average masonry material properties

Parameter	Mortar compressive strength	Brick compressive strength	Masonry compressive strength	Masonry bond strength	Cohesion	Coefficient of friction
	f_{mc}	f_{bc}	f_{m}	f_{t}	c	μ
Strength	2.6 MPa	27.0 MPa	15.0 MPa	0.404 MPa	0.2 MPa	0.48 MPa
COV	22 %	13 %	19 %	32 %	15 %	16 %
Method of Test	ASTM C109/C109M-02	ASTM C67-03a	ASTM C1314-03b	ASTM 1072-05	NZSEE guidelines	NZSEE guidelines

5 TEST SET-UP AND APPARATUS

The wall was tested as shown in Figure 4. The horizontal shear force was applied at the top of the wall through a hydraulic-powered jack. The laboratory strong-wall was used as a reaction point. The wall was built directly onto the strong-floor, and over the two bottom courses lateral restraints were bolted down to stop the wall from sliding on the artificially smooth strong floor. The smooth surface of the strong-floor was considered as an artificial weak point, and by restraining the wall over the two bottom courses sliding on that bottom surface was eliminated. This enabled the possibility of a sliding failure occurring at a brick-brick interface, rather than at a brick-strong floor interface. In a real wall the base course is normally laid on a concrete footing, and it was deemed that the bond between the mortar and strong-floor did not accurately represent this. A steel loading beam was placed on and mortared to the top of the wall, transferring loads through friction into the top of the wall in both push and pull cycles. This arrangement approximated the diaphragm-wall earthquake transfer mechanism in a URM building. Unbonded post-tensioning tendons on the outside of the wall were used to effect the axial load through the top of the wall. This axial load was kept constant throughout the test. An independent frame was used at one end of the wall against which to measure displacements. This was to eliminate any effects from flexing of the strong-wall. Displacement was measured at the tip of the wall on the opposite end from the loading jack.

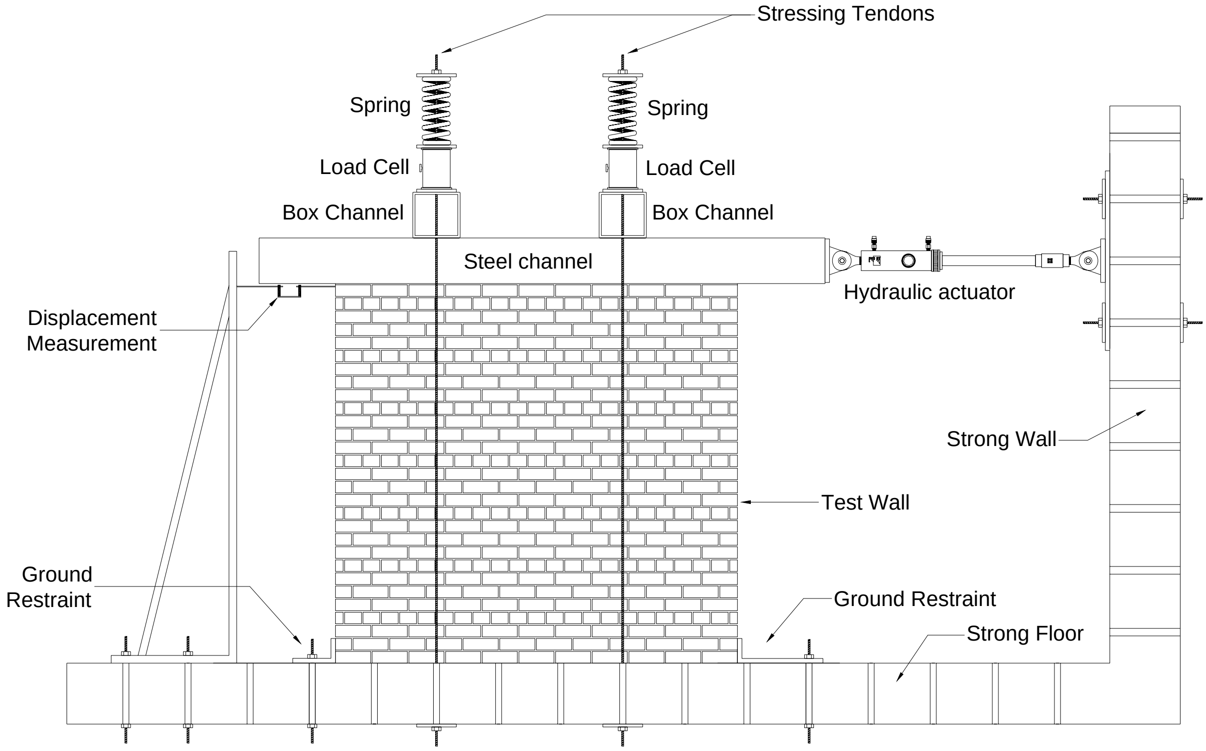


Figure 4: Test setup

5.1 *Loading History*

The loading history is shown in Figure 5. This displacement-controlled pseudo-static procedure was employed to capture the non-uniformly accumulated damage in the wall, and to enable observations of damage and failure mechanisms. The wall was subjected to displacements of 1 mm (0.04 % drift) in each direction and then the displacements were increased by increments of 1 mm each cycle until 10 mm was reached. Cracking was initially observed at 3 mm displacement. From displacements of 10 mm – 32 mm the increments were 2 mm (0.08 % drift) each cycle, and from 32 mm – 40 mm the increments were 4 mm (0.16% drift). From 40 mm – 50 mm the increments were 5 mm (0.2 % drift) each cycle, and the test was terminated at 50 mm wall tip displacement, corresponding to a wall tip drift of 2.0 %.

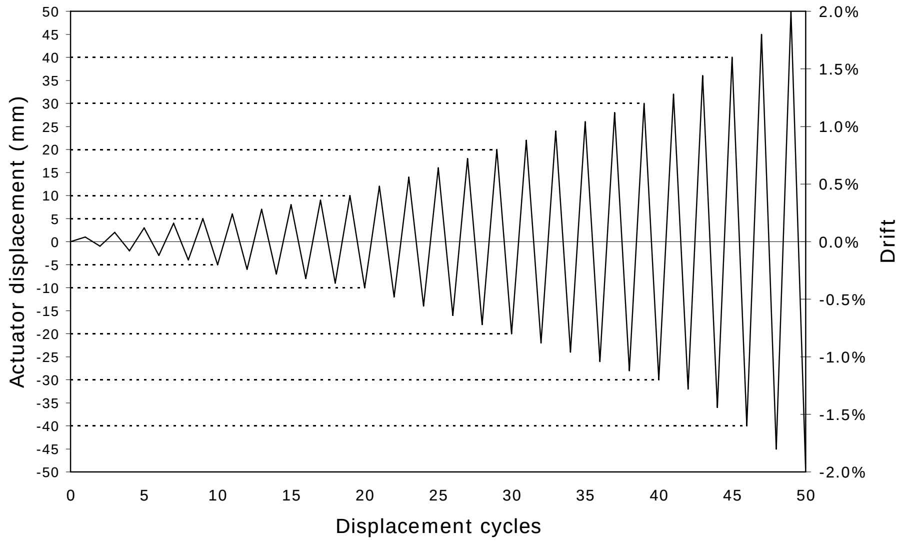
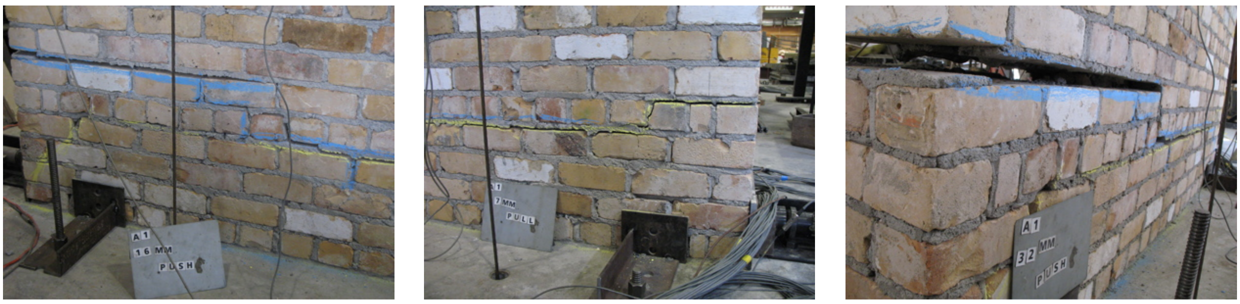


Figure 5: Pseudo-static displacement-controlled loading history employed in Wall A1

6 TEST RESULTS

Initial cracking was observed at a drift of 0.12 % (displacement 3 mm) and the corresponding base shear was 23.7 kN. A single crack opened up on the push cycle between the 6^{th} and 7^{th} courses and extended to approximately the mid-point of the wall. During the pull cycle to a drift of 0.12 % (displacement 3 mm), a similar crack was observed on the opposite end of the wall between the 4th and 5th courses, also extending approximately half the length of the wall, and corresponded to a base shear of 21.9 kN. In subsequent cycles these cracks propagated towards the other end of the wall until they extended approximately 2100 mm from each side, for the appropriate push or pull cycle. Wall rocking was observed and occurred on a contact area approximately 400 mm long (and across the full width of the wall) at each end. Figure 6 shows the cracks which opened up at the end of the wall. At a drift of 0.8 % it was determined that rocking was the overall failure mode. The cracks originating from each end met in the middle, and cracking through the header joints in two courses joined the cracks between the different courses. Some sliding occurred on the bedjoint cracks which had opened from rocking. At a drift of 0.4 %, compression cracks were observed in the toe crushing region of the wall. An increase in lateral force was observed after initial cracking as the displacement increased.



(a) uplift on push cycle (b) uplift on pull cycle (c) uplift on push cycle

Figure 6: Rocking response of test wall

The wall behaved elastically to a drift of approximately 0.12 %, after which rocking ensued and a small increase in load was observed for the corresponding increase in displacement. The force-displacement envelope is shown in Figure 7 (the full hysteresis loops are not shown, due to an error in the laboratory data-logging equipment). It is thought that the increase in load after lift-off of the wall can be attributed to an increased force in the post-tensioning tendons as they elongated, which were used on the outside of the wall to provide the axial load. There was very little observed damage apart from the crack where the wall was lifting off, and some very small cracks in the compression region.

It has been reported by other researchers (Abrams 1999) that a wall drift of 1.0% corresponds to exceeding the limit state of collapse prevention (CP). FEMA 356 (2000) gives performance levels for primary URM structures of immediate occupancy (0.1 %), life safety (0.3 %) and collapse prevention (0.4 %); and for secondary structures, life safety (0.6 %) and collapse prevention (0.8 %). There was no collapse of this wall even when the test was terminated at a drift of 2.0 %.

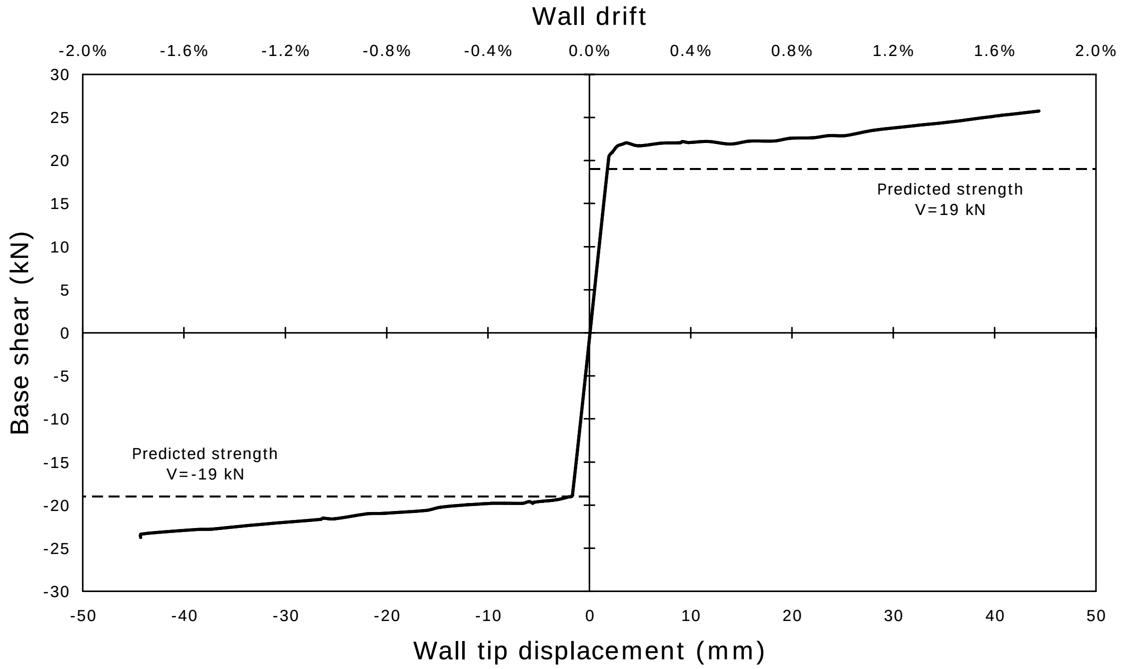


Figure 7: Force-displacement envelope

7 METHODS FOR PREDICTING WALL STRENGTH

The New Zealand Society for Earthquake Engineering (2006) provides guidelines for predicting wall strength. The strength limits based on sliding shear (V_s), damage to mortar in joints near points of contraflexure (V_j), diagonal tension failure (V_b) and flexural resistance (rocking, V_r) are given in Table 3. These values are derived using material data as shown in Table 1. Equations for predicting strength using FEMA 356 (2000) are also given in Table 3, for bed-joint-sliding (V_{bjs}), toe crushing (V_{tc}), diagonal tension (V_{dt}) and rocking (V_r). Symbols used in these expressions are given in Table 2.

Table 2: List of symbols

Symbol		Units	Symbol		Units
N	Normal force on cross-section	N	d	Depth of member	mm
N_D	Superimposed dead load at top of wall	N	c	Cohesion	N/mm^2
α	Factor equal to 0.5 for fixed-free cantilever, or 1.0 for fixed-fixed pier		f_{bt}	Direct tensile strength of bricks	N/mm^2
α_c	Effective aspect ratio		t	Thickness of wall	mm
f_{m}	Compressive strength of masonry	N/mm^2	L	Length of wall	mm
f_{a}	Axial compressive stress due to gravity loads	N/mm^2	h_{eff}	Height to resultant of lateral force	mm
z	Distance from extreme compression fibre to line of N	mm	v_{me}	Cohesive strength of masonry bed joint	N/mm^2
A_n	Area of net mortared section	mm^2	μ	Coefficient of friction	N/mm^2

Table 3: Predicted wall strengths

Failure mode	Sliding	Mortar damage (Toe crushing)	Diagonal tension failure	Rocking (Flexure)
NZSEE (2006)	$V_s = \dfrac{3czt + \mu N}{1 + \dfrac{3\alpha_c cdt}{N}}$	$V_j = \dfrac{cdt + \mu N}{1 + \alpha_c}$	$V_b = \dfrac{\sqrt{f_{bt} dt (f_{bt} dt + N)}}{2.3(1 + \alpha_c)}$	$V_r = \dfrac{N}{d}\left(z - \dfrac{1}{2}\dfrac{N}{0.85 f_c t}\right)$
Predicted strength	18.9 kN	99.1 kN	415.0 kN	19.4 kN
FEMA 356 (2000)	$V_{bjs} = v_{me} A_n$	$V_{tc} = \alpha N \left(\dfrac{L}{h_{eff}}\right)\left(1 - \dfrac{f_a}{0.7 f_m}\right)$	$V_{dt} = f_{dt} A_n \left(\dfrac{L}{h_{eff}}\right)\sqrt{1 + \dfrac{f_a}{f_{dt}}}$	$V_r = 0.9 \alpha N \left(\dfrac{L}{h_{eff}}\right)$
Predicted strength	171.0 kN	19.4 kN	251.9 kN	17.7 kN

The actual onset of cracking and failure due to rocking occurred at a load of approximately 24 kN, and it was predicted that rocking or sliding would govern, and occur at a load of approximately 19 kN, using NZSEE guidelines and 18 kN using FEMA 356 guidelines. The mode of failure was successfully predicted by both NZSEE and FEMA.

8 CONCLUSIONS

The wall reported here failed in rocking. This mode of failure was predicted by both NZSEE and FEMA guidelines. The following conclusions have been drawn:

- It will be useful to implement performance criteria for assessing New Zealand URM buildings. Although the wall initiated rocking approximately as predicted, it did not become unstable after that force was reached. Some performance criteria would allow the stability of a component to be utilized after the limiting force has been attained, especially when the limiting failure mode is rocking behaviour.
- Drift limitations as defined in FEMA 356 could be modified and/or developed for a New Zealand context.
- There are some differences between the limiting strengths produced by FEMA 356 and NZSEE guidelines.
- Further research into different boundary conditions will be conducted to investigate system level response of New Zealand URM walls, particularly to research the effect of connections between in-plane and out-of-plane walls, and if flanges allow rocking as a failure mode.

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