FLANGE EFFECTS OF AN UNREINFORCED MASONRY WALL SUBJECTED TO PSEUDO-STATIC IN-PLANE SEISMIC FORCES

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ABSTRACT :

Many unreinforced masonry (URM) buildings in New Zealand can be expected to perform poorly in an earthquake. The construction of such buildings was common in the early part of the 20th Century, but design philosophies focused on gravity loading, with little thought given to the lateral force resistance of brick walls. Consequently many URM buildings form a significant part of both New Zealand's heritage building stock and that group of buildings which are considered earthquake prone. As part of a programme of research to provide retrofit solutions for upgrading such buildings it was determined that the first task to be completed was the development or adoption of accurate seismic assessment techniques. Towards this aim, testing has been conducted to investigate the seismic in-plane performance of unreinforced masonry walls. Walls were constructed and testing was conducted to investigate the effects of boundary conditions, and in particular the effects of wall flanges. It was found that existing simplified predictive techniques do not accurately take into account the effects of boundary conditions, in particular the influence of perpendicular walls, to the failure mode of in-plane loaded walls. The effects of flange walls reduced the force and displacement capacity of the in-plane loaded wall, and also changed the failure mode from rocking for an unrestrained wall to shear cracking for the flanged wall.

KEYWORDS: Unreinforced masonry, Flange effects, Seismic assessment

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 demolished (DBH 2007; NZSEE 2006). 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; Magenes and Calvi 1997; Manzouri et al. 1996). 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 (NZSEE 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 2001; Abrams and Costley 1996; Paquette and Bruneau 1999). 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 boundary conditions of an experimental isolated wall have a strong influence on how well the wall represents system level response.

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.

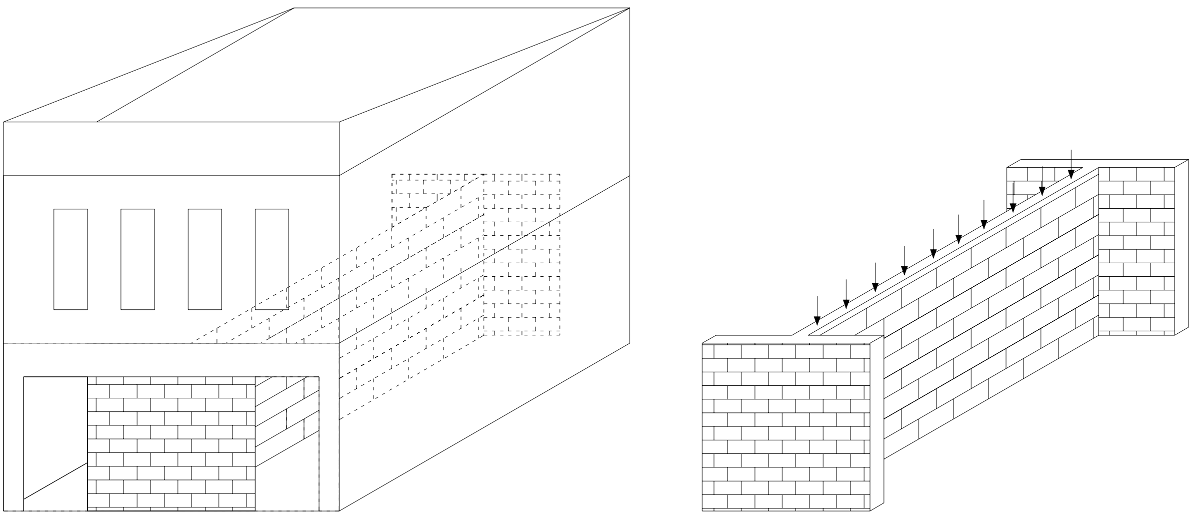
The laboratory test reported in this paper is one from a series of tests aimed at developing a comprehensive appreciation of the seismic performance of URM structures in New Zealand. An understanding of the expected performance in terms of material properties, component level response and system level response will be gained. This will provide a platform against which to compare the building seismic performance of retrofitted URM structures, and facilitate the implementation of cost-effective and economically viable seismic retrofit solutions. Also, the influence of components on the overall building response will be investigated throughout this overall experimental programme. 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 2002; Blaikie and Spurr 1992). Guidelines on the assessment of in-plane wall response (NZSEE 2006) have mainly been derived from results obtained overseas (Foss 2001; Magenes and Calvi 1997; Magenes and Della-Fontana 1998). 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 two-leaf (wythe) wall was constructed using recycled bricks. The wall simulated a central wall in a two storey isolated URM structure, as shown in Figure 2. This type of structure is common in New Zealand (Russell and Ingham 2008).



(a) prototype building (b) test wall

Figure 2: Test wall simulated one extracted from a 2-storey URM building

The test structure was a single in-plane URM wall, with a “flange” wall at each end. The two “flange” walls were perpendicular to the main wall, and acted as out-of-plane walls connected to the main wall. The in-plane wall measured 1500 mm long × 2000 mm tall × 240 mm thick (two leaves). The flange walls measured 1200 mm long × 2000 mm tall × 240 mm thick (two leaves). Thus the length of the overall structure was 1980 mm. The bond pattern employed was Common (American) bond, with header bricks every 4th course. 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 10 kN (11 kPa) was applied to the top of the wall, to simulate the axial load experienced on the central wall of a characteristic New Zealand two-storey isolated URM building, as shown in Figure 2. A timber diaphragm was built on top of the structure, with joists at 250 mm c/c and plywood on top (see Figure 5). The diaphragm was intentionally made stiff, and its purpose was to enable the horizontal force to be distributed (among the joists), instead of being applied at a single point in the top corner of the test wall. The diaphragm was also a platform on which to place weights for the application of axial (gravity) load.

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. 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. 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.

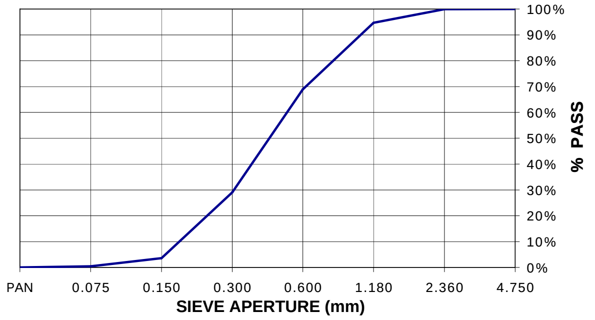


Figure 3: Mortar sand gradation curve

Table 1: Average masonry material properties

Parameter	Mortar compressive strength	Brick compressive strength	Masonry compressive strength
	f_{mc}	f_{bc}	f_{m}
Strength	13.7 MPa	27.0 MPa	18.1 MPa
COV	13 %	13 %	19 %
Method of Test	ASTM C109/C109M-02	ASTM C67-03a	ASTM C1314-03b

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. A steel loading beam was connected to the timber diaphragm using coach screws. The hydraulic jack was connected to the steel beam, which transferred the horizontal force into the diaphragm, and then the joists transferred the force into the brick wall. 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.

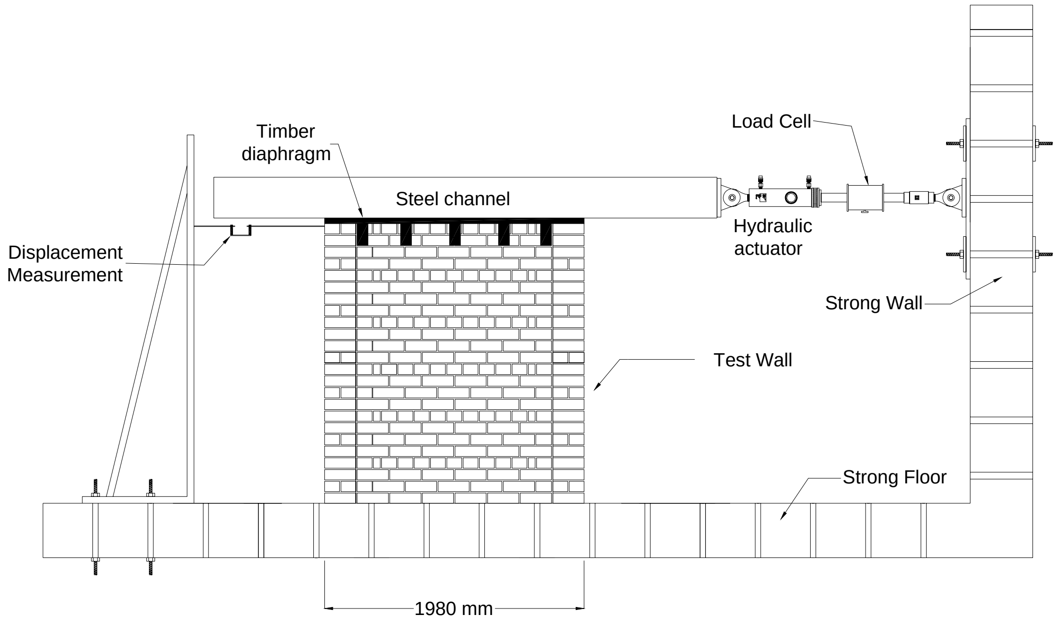


Figure 4: Test setup

The horizontal force was supposed to be applied as an imposed displacements, increasing incrementally in each direction throughout the test. That is, 1 mm push, 1 mm pull, 2 mm push, 2 mm pull, and so on until failure. However the wall failed in shear at a wall displacement of 0.4 mm push, and the equivalent of 0.4 mm in the pull direction.

6 TEST RESULTS

The wall was pushed to a displacement of 0.4 mm and a large crack suddenly occurred. The occurrence of the crack corresponded with a force of 66 kN. The 0.4 mm of displacement was between the bricks in the top course and the instrumentation column used for measuring the displacement. This displacement was different than the displacement measured between the steel loading beam and the instrumentation column. The displacement of the wall was used as the control, and the imposed displacement was applied until this displacement reached 0.5 mm. The shear crack occurred when this displacement was at 0.4 mm, but the displacement of the loading beam was much higher, approximately 8 mm, see Figure 6. This difference in displacement was attributed to slip between the steel loading beam and the timber diaphragm, that is in the coach screws, or between the timber diaphragm and the wall, or it can be attributed to the flexibility of the timber compared with the masonry. The development of the crack was very sudden, and initiated at the location of the joist at the end of the wall, refer to Figure 5. It continued along the wall and into the flange, 4 courses below. The load was then released and the crack closed up and the wall returned to its original state, albeit with a crack in it.

The load was then applied in the pull direction. When the load from the push cycle was released there was a residual displacement of 1 mm. The wall was pulled and a crack opened up in a similar manner to what happened in the push cycle. This crack occurred when the control displacement was reading 0.67 mm, so there was a net pull of 0.33 mm before the crack occurred. Again, this was the displacement of the bricks, and the displacement of the loading beam was much larger, approximately –7 mm, resulting in a net displacement of 8 mm. The crack was almost identical to the one which opened up on the push cycle, in reverse, and initiated at the point where the first joist was in the wall.

(a) crack in flange wall

(b) shear crack push cycle

(c) shear crack pull cycle

(d) shear crack pull cycle

Figure 5: Shear failure of test wall

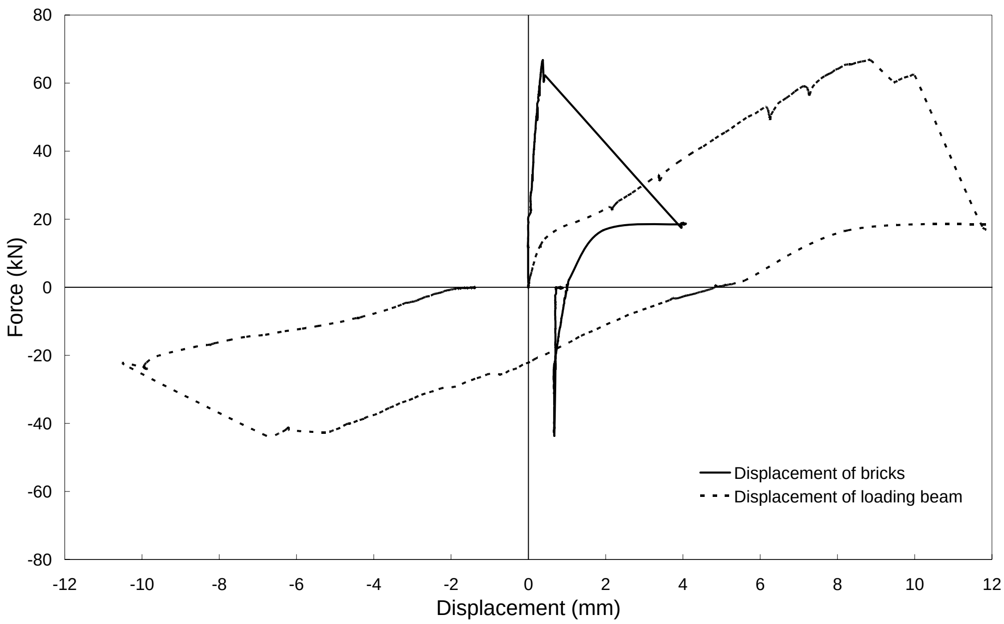


Figure 6: Force-displacement response

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. Both FEMA 356 and the NZSEE guidelines predicted that rocking would be a likely limiting failure mode, and sliding would be possible also. A limiting strength due to shear failure was predicted to be very unlikely by both methods. Both methods of assessment failed to accurately predict the mode of failure, because the presence of flanged walls was not taken into account.

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)	$V_s = \dfrac{3czt + \mu N}{1 + \dfrac{3\alpha_c cdt}{N}}$	$V_j = \dfrac{cdt + \mu N}{1 + \alpha_c}$	$V_b = \dfrac{\sqrt{f_{bt}dt\left(f_{bt}dt + N\right)}}{2.3\left(1 + \alpha_c\right)}$	$V_r = \dfrac{N}{d}\left(z - \dfrac{1}{2}\dfrac{N}{0.85 f_c t}\right)$
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

8 CONCLUSIONS

- The wall failed in shear, but neither of the guidelines commonly used in New Zealand to predict wall response predicted that shear would be the failure mode.

- The boundary conditions (in this case the flange walls) were not adequately accounted for in the prediction of the wall strength. This resulted in an incorrect prediction of failure.
- Further research into boundary conditions needs to 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.

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