EARTHQUAKE-DAMAGED UNREINFORCED MASONRY BUILDING TESTED IN-SITU

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ABSTRACT

In December 2007 a magnitude 6.8 earthquake had an epicentre located approximately 50 km from the city of Gisborne, New Zealand. This earthquake caused damage to a number of buildings in Gisborne, and in particular, to numerous unreinforced masonry buildings. One such building was damaged to the extent that significant post-earthquake repairs were necessary, and partial removal of two of the building's gable ended walls was required. This reconstruction provided an opportunity for a team of researchers from the University of Auckland to conduct field tests on the building, allowing comparison with companion testing that had previously been undertaken in a laboratory setting. This field testing involved the extraction of clay brick and mortar samples, in-situ bed joint shear tests, diagonal shear tests on samples extracted from the gabled walls, an in-situ in-plane shear test and out-of-plane testing of a gable ended wall both in the as-built condition and after the installation of a near-surface mounted (NSM) carbon fibre reinforced polymer (CFRP) retrofit solution. Testing confirmed that the boundary conditions in real buildings can significantly affect experimental response, with vertical restraint resulting in a large increase in out-of-plane load capacity, and also confirmed that the near-surface mounted FRP solution is an excellent low-invasive option for seismic strengthening of unreinforced masonry walls. Details of the history of the building, and the methods used to undertake the field testing are reported, and experimental results are presented.

1.0 INTRODUCTION

Unreinforced masonry (URM) was one of the most common construction materials in New Zealand prior to the 1931 Hawke's Bay earthquake (Dowrick 1998). The popularity of this form of construction has resulted in numerous URM buildings remaining throughout New Zealand, many of which are now considered to have significant national heritage value (Russell 2010). This type of construction often has insufficient strength to resist lateral earthquake forces in high and moderate seismic zones and lacks the ability to dissipate energy.

On 20 December 2007 at 8:55 pm, a Richter magnitude 6.8 earthquake occurred with an epicentre located approximately 50 km from the city of Gisborne (GNS 2008). Although there were no directly related fatalities, the earthquake caused damage to a number of buildings in Gisborne's central business district (CBD), which has a collection of historic buildings dating from the 1880s that are predominantly built of unreinforced masonry and timber. There were numerous instances of cracked walls and partial collapse, particularly in unreinforced masonry buildings, but most houses and commercial buildings were not significantly damaged (McClean and Wallace 2009). It has been estimated that up to 90% of the earthquake damage was attributed to over-topped parapets in unreinforced masonry buildings (Petty 2008). One such building, the B-East building of the Allen's Trade Complex, was damaged to the extent that significant post-earthquake repairs were necessary, which required partial removal of two gable ended walls.

As most research considering seismic assessment of URM walls has been conducted using laboratory-based studies with artificial boundary conditions, in-situ testing is an important opportunity to provide data to validate the accuracy of laboratory-based studies on wall behaviour. In 2009, the reconstruction of the subject building and the removal of the aforementioned earthquake

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damaged end gables presented an opportunity for a team of researchers from the University of Auckland to conduct field tests on the building, allowing comparison with companion testing that had previously been undertaken in a laboratory setting. A number of tests were performed on the building on site, and later in the laboratory on extracted wall samples. In particular, this field testing involved the extraction of clay brick and mortar samples, in-situ bed joint shear tests, diagonal shear tests on samples extracted from the gabled walls, an in-situ in-plane shear test and out-of-plane testing of the gable ended wall both in the as-built condition and after the installation of a near-surface mounted (NSM) carbon fibre reinforced polymer (CFRP) retrofit solution. Test results and discussions are presented here.

2.0 DESCRIPTION OF TESTED BUILDING

The Allen's Trade Complex has had a number of varied uses throughout its life, ranging from a storage warehouse to a mechanical workshop and a gathering center for a community trust. The complex is comprised of five main buildings, one of which suffered significant structural damage during the 2007 Gisborne earthquake. Testing was performed on the earthquake damaged B-East building facing Pitt Street (refer Figure 1) that was originally constructed in 1911. In 1984 the B-East building was registered as a heritage building with the New Zealand Heritage Places Trust (NZHPT), and as such the option of complete demolition of the building was not considered, and any strengthening and securing works had to be sensitive to the existing building fabric and be designed in consultation with Gisborne District Council and the NZHPT.

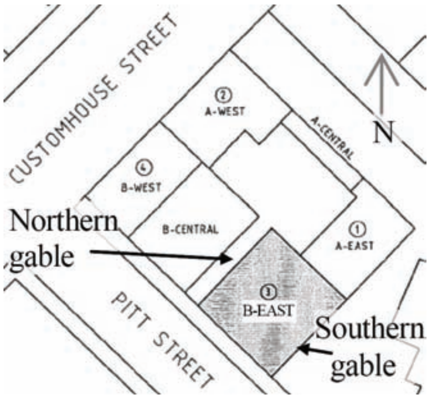


Figure 1. Allen's Trade Complex location map

The 2 storey building (shown in Figure 2) is located in the CBD of Gisborne, New Zealand, and was originally constructed of unreinforced masonry with perimeter walls comprised of 2 to 4 leaf thick solid masonry. The building has an almost square footprint of approximately 390 m^2. The first level floor diaphragm is constructed from timber joists and timber tongue and groove flooring and the roof of the building is a single gable constructed from timber framed trusses clad with corrugated iron sheets supported on wooden rafters. Both the roof and the floor diaphragms are connected to the perimeter walls using through steel anchors. The main masonry type for the original building consisted of red coloured soft bricks with soft lime/cement mortar.

(a) View facing north

(b) View facing north east

Figure 2. Allen's Trade Complex B-East building

The B-East building was originally sandwiched between the existing timber framed building adjacent to the northern end gable (sharing the wall with B-East building) and a demolished building that used to be adjacent to the southern end gable. Although the exact date of demolition is uncertain, the removal of the adjacent building left the masonry of the southern end gable exposed to the environment for over a decade. There were two concrete ring beams extending along the perimeter of the building, one at approximately mid-height of the ground floor level and the second at approximately mid-height of the first floor level.

2.1 The Gisborne Earthquake Damage

During the 2007 Gisborne earthquake the B-East building suffered extensive structural damage (shown in Figure 3) to the extent that significant post-earthquake repairs were necessary and the removal of two gable ended walls was required. As a result the building was designated unstable and unsafe to occupy until reconstruction took place. Following the 2007 Gisborne earthquake, numerous visible cracks in the vicinity of the window piers and wall corners were observed. As shown in (a) and (b) in the Figure 3 the top portion of the northern gable corner separated from the cross wall, causing the entire top portion of the southern gable to permanently deform outwards from the roof diaphragm. The northern gable sustained less damage, with only a small number of visible cracks. Due to instability and safety considerations, the top portion of the southern end gable required demolition prior to the commencement of any testing. There were a number of locations where the anchors connecting the top of the gable to the roof diaphragm were pulled out from masonry, as shown in Figure 3(c).

(a) Cracking of the top gable corners

(b) Cracking of the top gable corners (internal view)

(c) Roof anchor pull-out

Figure 3. B-East building damage due to 2007 Gisborne earthquake

2.2 Description of Test Locations

Testing was performed only on the earthquake affected end gables which were scheduled for demolition, and occurred in two separate stages. The first stage of testing focused primarily on in-plane wall response and was conducted on the southern gable. Prior to the commencement of testing the unstable 2 leaf portion of the gable and the upper concrete ring beam had been removed. The wall structure below the unstable portion of the gable consisted of 3 leaf thick masonry and 'masonry columns' (with increased wall thickness to 4 leafs) spaced at approximately 3000 mm. The individual test locations were selected in the 3 leaf thick masonry wall only, and the 4 leaf thick masonry columns were avoided. The second stage of testing focused on the northern gable which, apart from a few cracks, was mostly undamaged during the earthquake. This gable consisted of a 2 leaf thick masonry wall and had a cement/lime plaster layer on the exterior surface. A mezzanine floor positioned 2500 mm above the first floor level was adjacent to both gables.

3.0 MATERIAL PROPERTIES

The material properties of B-East building were determined through in-situ testing and laboratory tests on samples that were extracted from walls scheduled for demolition. From visual observation, the constituent materials of the building were of average quality. The mortar joints from the northern gable were less weathered and slightly darker in colour than those from the southern gable, suggesting that the mortar from the northern gable had higher cement content.

Figure 4 illustrates the masonry type resident within the wall that was subjected to in-situ tests and sample extraction. This type of masonry was uniform throughout the building, and consisted of 75 mm × 220 mm × 105 mm red solid clay bricks, and 12 mm thick cement-lime mortar. Irregular mortar samples, single bricks and three brick high masonry prisms were extracted from the building site and tested in the laboratory.

3.1 Brick Compressive Strength

The brick compression strength was obtained using the half brick compression test method ASTM C 67-03a (ASTM, 2003). The average half brick compressive strength, f'_b, and the coefficient of variation are presented in Table 1. The bricks were classified as stiff bricks based on their physical appearance, according to the New Zealand Society for Earthquake Engineering (NZSEE 2006) guidelines for seismic assessment and strengthening of buildings.

Table 1. Average half brick compressive strength (f'_b) and NZSEE (2006) suggested values

Sample Size	f'_b (MPa)	Range (MPa)	CoV	NZSEE recommendation (MPa)
9	19.4	16.5–26.1	0.158	10 – 20

The compressive strength of the bricks was found to be between 16.5 and 26.1 MPa with an average of 19.4 MPa, which is in good agreement with the NZSEE recommendation for the corresponding brick type.

3.2 Masonry prism compressive strength and Modulus of Elasticity

Six 3-brick high masonry prisms were extracted from the northern gable of the B-East building and tested in compression in accordance with ASTM C 1314-03b (ASTM 2003a). Displacement gauges were incorporated in the masonry prism testing setup to obtain the stress-strain response and to calculate the Modulus of Elasticity, which was obtained as the slope of the stress-strain curves between 0.05 and 0.70 times the maximum prism compressive strength (f'_m). The average compressive strength and Modulus of Elasticity value (E) of the samples are presented in Table 2.

Table 2. Average masonry prism compressive strength (f'_m) and Modulus of Elasticity (E)

Sample Size	f'_m (MPa)	f'_m CoV	E (GPa)	E CoV	E/f'_m
5	9.6	0.32	2.35	0.14	279

3.3 Mortar bed joint shear strength

The mortar bed joint shear strength was determined on-site using the in-situ shear test procedure ASTM C 1531-03 (ASTM 2003b) as shown in Figure 4. A total of ten in-situ shear tests were performed, with four tests performed on the northern gable and six tests performed on the southern gable. Flatjack devices were then used to apply axial precompression load during tests 2, 3, 5, 6, 9 and 10. The shear test results are presented in Table 3.

The data reported in Table 3 indicates that the average mortar bed joint shear strength of the southern end gable was significantly lower than that of the northern end gable.

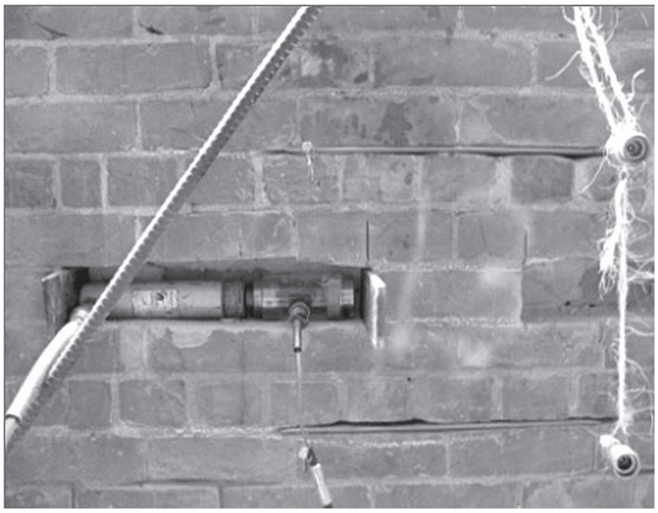


Figure 4. Typical mortar bed joint shear test setup

Table 3 . In-situ shear test results

Test Number	Location	Axial Precompression (MPa)	Shear Strength (MPa)
1	SG - exterior	0.016	0.15
2	SG - exterior	0.082	0.24
3	SG - exterior	0.180	0.25
4	SG - interior	0.007	0.16
5	SG - interior	0.230	0.13
6	SG - interior	0.310	0.19
7	NG	0.059	0.90
8	NG	0.080	0.94
9	NG	0.390	0.90
10	NG	0.470	0.69

Note: SG – southern gable, NG – northern gable

The significant difference in the mortar bed joint shear strength was attributed to a higher level of deterioration of the mortar joints in the southern gable than that in the northern gable and to the possible variation in mortar mixes during wall construction. It was observed that the bed joint shear strength was not significantly affected by variations in the axial precompression stress.

For this building, the cohesion values were determined to be adequately described as the average of shear strengths at different levels of axial precompression for each wall, and therefore are equal to 0.86 MPa and 0.19 MPa for the northern and southern gables respectively.

4.0 EXPERIMENTAL PROGRAM

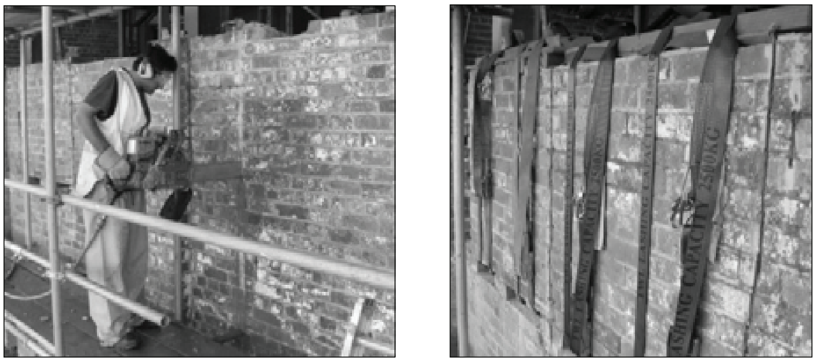
On site testing was divided into two stages to reduce the interruptions for the operating construction company. The first stage of site testing involved the southern gable only, and investigated masonry in-plane behavior. In order to accurately determine the diagonal masonry shear strength and to provide direct comparison to laboratory-built wall panels, six wall panels having approximate dimensions of 1200 mm × 1200 mm were cut, extracted and transported to the University of Auckland laboratory for testing. Due to time restrictions on site, only one wall panel was tested in diagonal shear in-situ (data not included due to equipment malfunction). To further investigate the in-plane strength capacity and the failure mechanism of the existing masonry, a push over test was performed on a 2600 mm long × 1300 mm high × 3 leaf 330 mm thick wall section.

The second stage of site testing involved the northern gable and was focused mainly on investigating masonry out-of-plane behavior. The particular focus was to conduct one way out-of-plane bending tests on an as-built wall strip that was separated from a larger wall section. Following the completion of the as-built test the test wall was retrofitted (repaired) using the Near Surface Mounting (NSM) technique with a single CFRP strip embedded into the masonry. The purpose of the repair was to observe the effects that the NSM retrofit technique had on wall strength and stiffness.

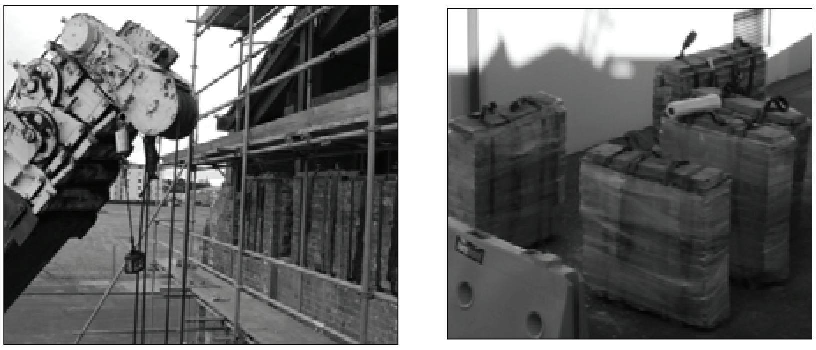
5.0 WALL PANEL EXTRACTION AND DIAGONAL SHEAR TEST

5.1 Extraction Procedure

The locations of the wall panels were carefully selected to minimise the required length of masonry cutting, the cutting locations were clearly marked, and the vertical cuts were made using a concrete cutting chainsaw. In order to extract the wall panels, two through penetrations per panel were made to allow lifting straps to be used. To prevent the panels from cracking or breaking during lifting and/or transportation, each wall panel was vertically strapped with two heavy duty ratchet tie downs. This secured the masonry together by applying axial force to minimise the risk of damage. Once the cutting and securing work was completed the wall panels were lifted out of the building and onto a truck using a medium sized crane, and transported to the University of Auckland in an undamaged condition. Wall panel preparation and extraction is shown in Figure 5.



(a) Cutting through masonry **(b) Vertical straps**



(c) Lifting wall panels using a medium sized crane **(d) Wrapping wall panels with protective plastic**

Figure 5. Wall panel preparation and extraction

5.2 Laboratory Diagonal Shear Test

The wall panels were tested in diagonal shear (in accordance with ASTM E 519 – 07) (ASTM 2007). The testing procedure involved rotation of the URM wall panel by 45° and vertical loading along one of the wall's diagonals. For testing of the wall panels extracted from the B-East building the standard method was modified such that the wall remained vertical in its original orientation and the loading mechanism was rotated. The test setup is shown in Figure 6.

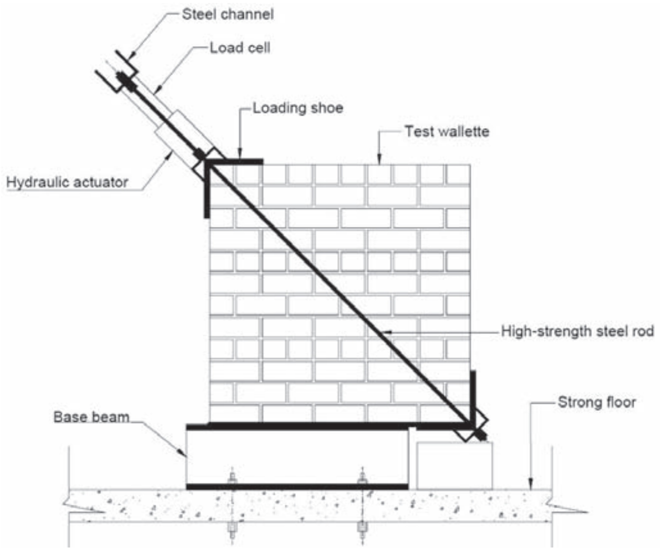


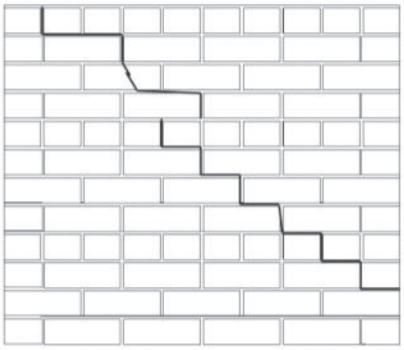
Figure 6. Diagonal shear test setup

5.3 Test Results

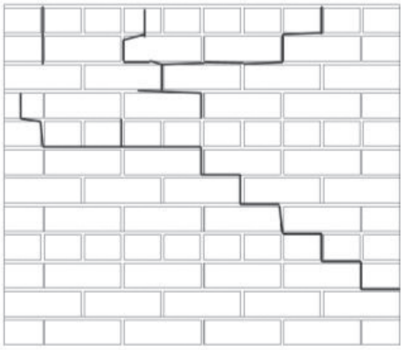
All wall panels failed by crack propagation through the mortar joints in a step-wise mode, with the final crack patterns illustrated in Figure 7. In order to provide direct comparison to the results obtained from laboratory-built wall panels, and to facilitate interpretation and use of the data by structural engineering practitioners, the wall panel displacements are reported as shear strain as shown in Figure 8. It is noted that test panel AGW4 (as-built Gisborne wall panel 4) suffered minor damage during the setup preparation in the laboratory, resulting in the sample being pre-cracked at the 7th brick course from the bottom. This is visible in the crack pattern shown in Figure 7(b) and is responsible for the reduction in shear stress at cracking that is illustrated in Figure 8(d).

5.4 Discussion

The diagonal shear strength varied among the tests, but within reasonable limits for URM construction. With the exclusion of the pre-damaged wall panel AGW4 the average shear stress at cracking was 0.057 MPa, with a maximum average shear stress of 0.082 MPa. The crack pattern observed at failure closely matched the pattern obtained in laboratory-built wall panels with weaker mortar mixes (Russell 2010). For all wall panels, with an exception of AGW3, the maximum shear stress was higher than the shear stress at cracking, which was not observed in laboratory-built samples where shear stress decreased immediately following cracking (Russell 2010). The results presented in Table 4 enable a comparison of the results from laboratory-built wall panels constructed using different mortar mixes to wall panels extracted from the B-East building. The results from wall panels manufactured in the laboratory with 1:2:9 mortar mix (cement:lime:sand) closely resemble values obtained from the B-East building wall panels. The average maximum shear stress for the laboratory-built wall panels using a 1:2:9 mortar mix was 0.094 MPa, which compares well to the 0.082 MPa mean value obtained from the B-East building wall panels. Laboratory built wall panels using a weaker mortar mix of 0:1:3 and a stronger mortar mix of 2:2:9 resulted in maximum shear stresses of 0.04 MPa and 0.50 MPa respectively, which were outside the range obtained from wall panels from the B-East building.

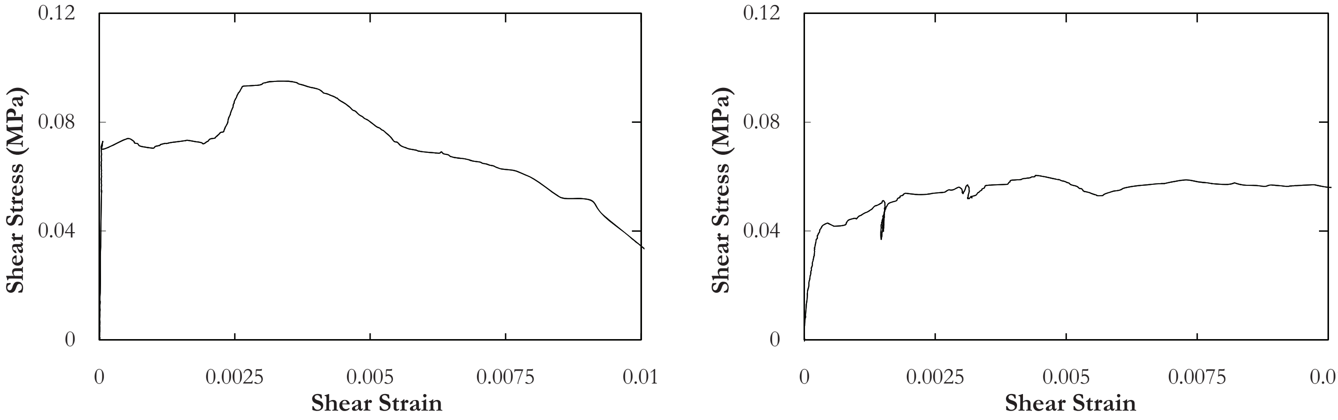


(a) Typical crack pattern



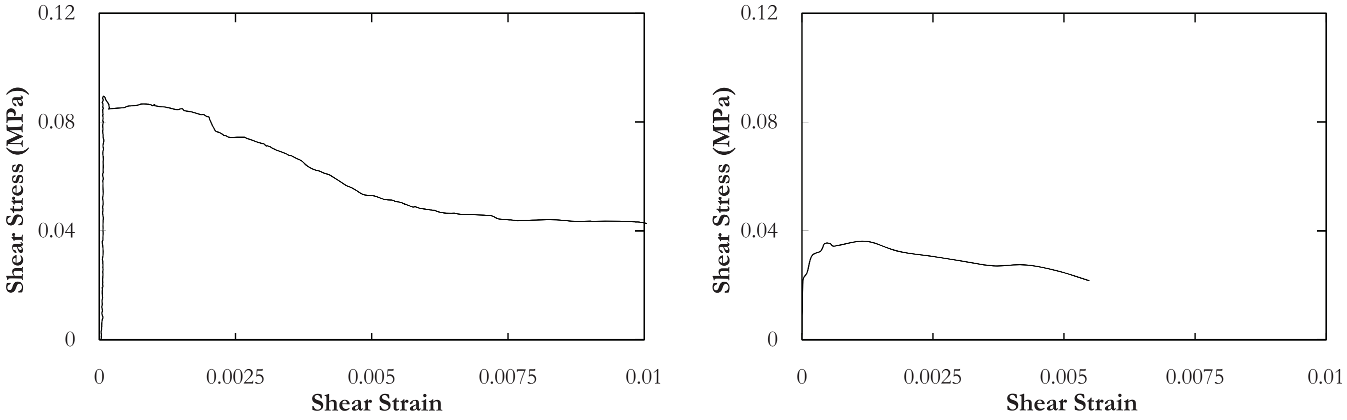
(b) AGW 4

Figure 7. Crack pattern



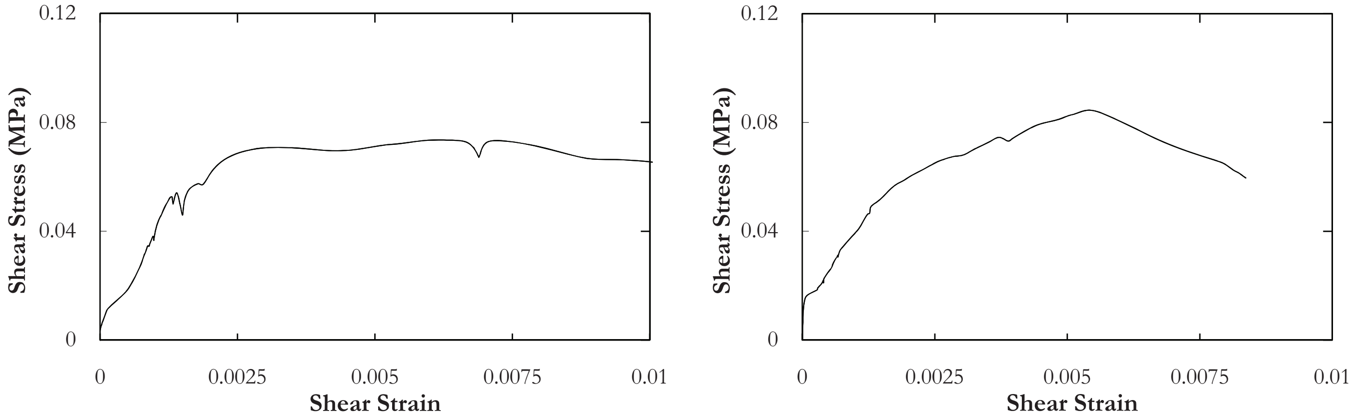
(a) Wall AGW 1

(b) Wall AGW 2



(c) Wall AGW 3

(d) Wall AGW 4



(e) Wall AGW 5

(f) Wall AGW 6

Figure 8. Shear response for wall panels in tested diagonal tension (shear)

Table 4. Summary of wall panel diagonal shear tests

Wall panel	Shear stress at cracking (MPa)	Shear strain at cracking	Maximum shear stress (MPa)	Shear strain at maximum shear stress
AGW1	0.073	0.0016	0.095	0.0034
AGW2	0.051	0.0015	0.060	0.0044
AGW3	0.090	0.0000	0.090	0.0000
AGW4	0.033	0.0018	0.036	0.0012
AGW5	0.054	0.0014	0.082	0.0240
AGW6	0.018	0.0003	0.084	0.0056
LAB1 (1:2:9)*	0.110	-	0.074	-
LAB2 (1:2:9)*	0.094	-	0.094	-
LAB3 (1:2:9)*	0.090	-	0.090	-
LAB4 (1:2:9)*	0.090	-	0.090	-
LAB5 (1:2:9)*	0.100	-	0.100	-
LAB6 (1:2:9)*	0.080	-	0.080	-
LAB7 (0:1:3)*	0.040	-	0.040	-
LAB8 (2:2:9)*	0.500	-	0.500	-

*- (1:2:9) refers to the ratio of cement:lime:sand (by volume) used in the mortar mix.

6.0 IN-PLANE PUSH TEST

In-plane response assessment for New Zealand URM walls currently relies on limited data obtained from laboratory-built walls in New Zealand, and from the NZSEE (2006) guidelines, which have primarily been developed using research data obtained from other countries. The objective of this in-situ push test was to acquire results to compare with corresponding data obtained from laboratory-based experiments.

6.1 Test Location and Setup Description

The test wall location was selected to have a maximum possible length, which was constrained by the location of masonry columns. The selected location was on the first level of the building as illustrated in Figure 9. The test section was isolated from the rest of the wall by vertical cuts on both sides of the section and had dimensions of 2.6 m long × 1.3 m (17 bricks) high × 0.33 m (three leaf) wide, with no additional axial load applied apart from the wall self weight. The wall was painted white to facilitate visual detection of formed cracks during the test.

Load was applied using a hydraulic actuator positioned at the top corner of the test section using the remaining larger part of the masonry wall as a reaction point, and a manual hydraulic pump was used to gradually increase loading of the test section. In-plane wall displacement was measured using 6 portal strain gauges positioned directly on the test section (as shown in Figure 10) to detect any possible rocking and diagonal deformations. Two linear variable differential transducers (LVDTs) were positioned at the top and bottom of the test section on the opposite end from the hydraulic actuator.

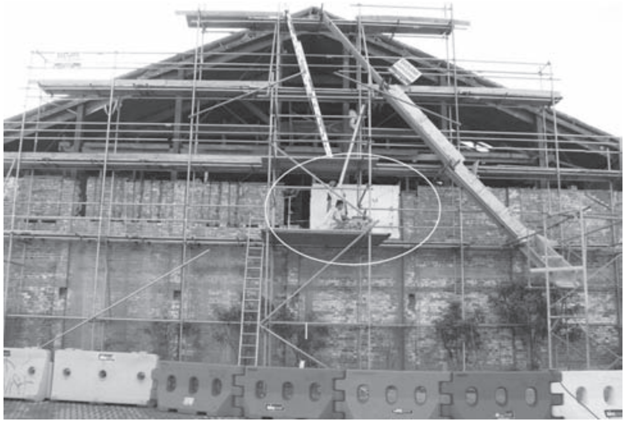


Figure 9. In-plane push test location, southern gable

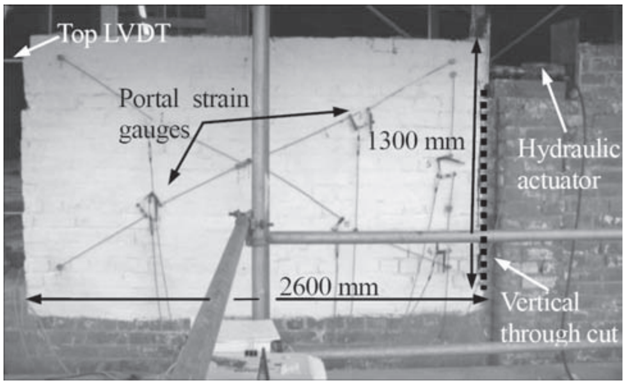


Figure 10. In-plane push test setup

6.2 Experimental Results

Initial cracking was observed at a displacement of approximately 2 mm (0.5% drift) with a corresponding base shear of 9.1 kN. With increasing loading the crack propagated from the loading side in a step-wise pattern through mortar joints and sliding of the section occurred, with the final crack pattern being as shown in Figure 11. The base shear-drift response is shown in Figure 12. Only the displacement values obtained from the top LVDT are reported here.

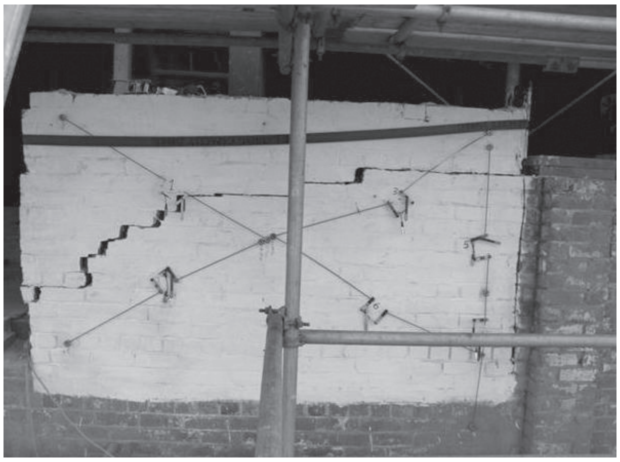


Figure 11. The crack pattern after push test

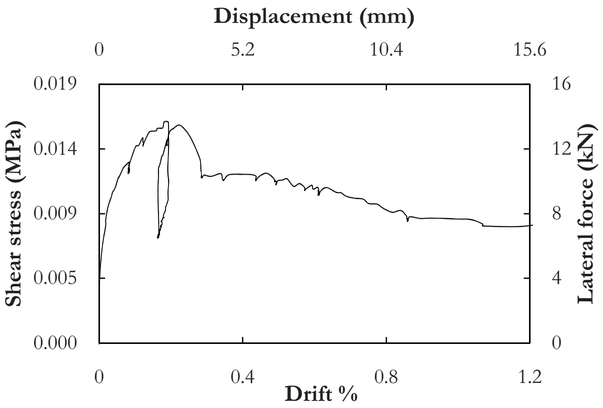


Figure 12. Base shear - drift response for in-plane push test

6.3 Discussion

During the test it was observed that the crack initiated at the loaded edge of the wall and gradually propagated to the opposite edge, indicating that the cohesion was engaged with the increasing load levels as indicated by the changing gradient between 0 and 0.2% drift for the curve shown in Figure 12. Based on simple static analysis, the applied force and the normal force due to the self weight of the test section acting on the sliding plane resulted in the coefficient of friction μ for the wall section being calculated to be 0.7.

In NZSEE (2006), the in-plane strength limit is based on sliding shear (V_s), damage in mortar joints near the point of contreflexure, diagonal tension failure and rocking. According to the observed failure mode in the wall panel, only the sliding failure mechanism is considered here. Equation 1 from NZSEE (2006) guidelines and Equation 2 from FEMA 356 (2000) are used to predict the sliding strength of the wall section. Symbols used in these expressions are given in Table 5.

$$V_s = \frac{3czt + \mu N}{1 + \frac{3\alpha_c cdt}{N}} \tag{1}$$

$$V_{bjs} = v_{me} A_n \tag{2}$$

Table 5. List of symbols

Symbol		Units
c	Cohesion	MPa
z	Distance from compression fibre to line of N	mm
t	Thickness of wall	mm
N	Normal force on cross-section	N
α_c	Effective aspect ratio	
μ	Coefficient of friction	
d	Depth of member	mm
v_{me}	Cohesive strength of masonry bed joint	MPa
A_n	Area of net mortared section	mm^2

Using material properties obtained from in-situ testing the sliding force was predicted to be 18.9 kN using NZSEE guidelines and 60 kN using FEMA 356. It is evident that both methods overestimated the force which initiated sliding.

7.0 OUT-OF-PLANE TESTS

7.1 Location and Setup Description

Two out-of-plane wall tests were conducted at the B-East building, one being in the as-build condition and one in the retrofitted (repaired) condition. A 2-leaf wall lined with 12-15 mm thick cement plaster finish on the exterior surface was selected for testing and was located within the southern gable as shown in Figure 13. The entire wall had dimensions of 19.8 m long × 10.8 m high with two door penetrations. In order to simplify the test and to provide possible comparison to laboratory-built specimens, a 1200 mm wide wall strip was isolated from the rest of the wall, inducing a one way bending failure. The wall strip was isolated by cutting vertically through the wall using a concrete cutting chainsaw, with vertical cuts made on an inward angle to eliminate wedging effects after wall deformations had occured. Due to the presence of the mezzanine floor the maximum possible height of the wall strip was restricted to 3000 mm.

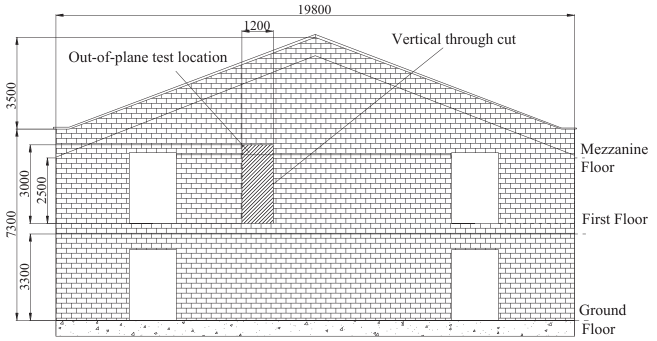


Figure 13. Southern gable elevation showing test location

Out-of-plane loading was applied by gradually inflating a 2.1 × 1.2 m Bigfoot™ vinyl airbag. To accommodate the airbag, a gap of 50 mm was left between the wall and the plywood backing. The plywood backing, measuring 2.4 m × 1.2 m, consisted of an assemblage of plywood sheets and steel angles and was supported by a reaction frame which consisted of vertical and diagonal timber members bolted to the timber floor joists to transfer horizontal load into the timber floor diaphragm. The cross-section of the setup is shown in Figure 14.

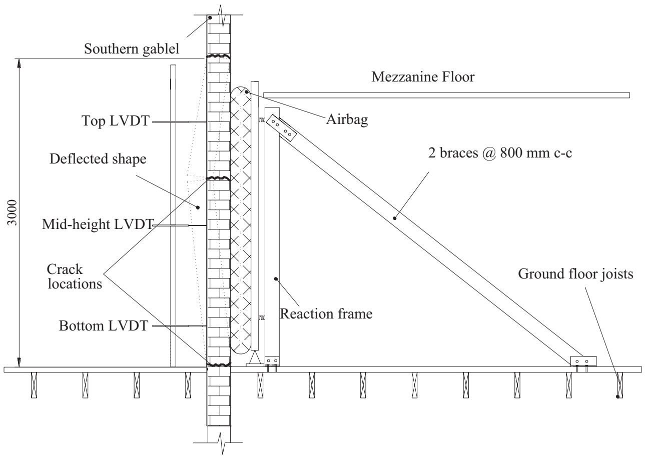


Figure 14. Cross-section details for out-of-plane test setup

The applied load from the airbags was transferred to the plywood backing and to the reaction frame using four S-type 10 kN load cells, which were attached between the plywood backing and the reaction frame and provided horizontal stability to the plywood backing frame. To ensure that the entire load was transferred through the load cells, frictionless plates were used underneath the plywood backing. The experimental setup is shown in Figure 15 and Figure 16(a) and closely resembled the setup successfully used previously (Derakhshan et al., 2010, Dizhur et al., 2009).

Out-of-plane displacement was measured using 3 LVDTs mounted on the opposite side of the wall. One LVDT was placed mid-height and at the wall centerline, and the second and third LVDTs were placed at 2600 mm and 600 mm above the floor level respectively, as shown in Figure 16(b). The data from load cells and LVDTs was collected at 50 Hz using a National Instruments data acquisition system.

Figure 15. Reaction frame for out-of-plane test

(a) Reaction frame close up **(b) LVDT locations**

Figure 16. Out-of-plane test setup

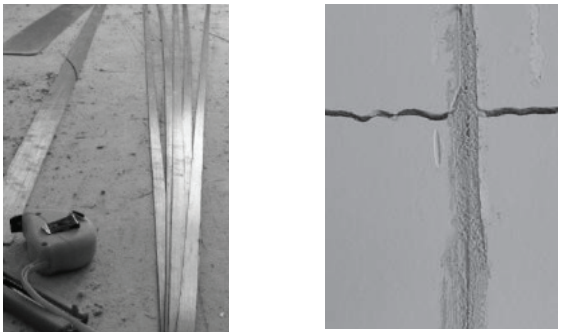
7.2 Retrofit Scheme

Using FRP material to retrofit URM walls is a technique for strengthening and increasing the ductility capacity of URM walls subjected to in-plane and out-of-plane earthquake loading. Externally bonded (EB) FRP sheets or plates and NSM FRP bars or strips are the two application techniques that are commonly used (Mosallam 2007, Yasser and Robert 2006). Using the NSM technique provides some protection from fire and the environment and if detailed correctly, does not adversely affect the aesthetics of the structure (Petersen and Masia 2008). In this field experiment, the simulated seismic performance of a NSM FRP strip was investigated.

Following the as-built test the wall test section was retrofitted (repaired) using a single 15 mm wide and 1.2 mm thick CFRP strip (Modulus of Elasticity of 165 GPa) positioned vertically in the centre of the wall test section and extending from top to bottom, as shown in Figure 17(b). A groove was cut into the brick through the plaster layer, using a diamond blade circular saw (5 mm thick blade) to a depth of approximately 30 mm to ensure direct bonding of the CFRP strip to the brick surface only. Two part epoxy adhesive was used to bond the CFRP strip into the groove. The groove was entirely filled with epoxy prior to CFRP strip insertion to ensure maximum bond area, and 24 hours were allocated for the epoxy to set. Figure 17 shows the installation procedure.

(a) Epoxy application

(b) Insertion of the CFRP strip



(c) 15 mm wide CFRP

(d) Close-up of NSM repair

Figure 17. CFRP retrofit application

7.3 Experimental Results

Two semi-cyclic out-of-plane tests on the as-built wall were first performed. The pressure in the airbag was slowly and uniformly increased, and the total force exerted by the airbag onto the wall was calculated by summation of the readings from the four load cells. The pressure in the airbag was increased until a horizontal crack located at 1600 mm above the floor level (at 53% of the wall strip height) formed at a displacement (mid-height) of approximately 5 mm. The pressure in the airbags was further increased until a displacement of approximately 50 mm was achieved, at which point the air pressure in the airbag was released. At zero face pressure a residual displacement of approximately 15 mm was observed. The pressure in the airbag was again gradually increased until a maximum displacement of 62 mm was recorded, after which airbag pressure was again released to zero. The total lateral load – mid-height displacement response for the as-built test is shown in Figure 18. At maximum loading pressure, evidence of wall movement at the top boundary was visually detected and additional vertical, horizontal, and diagonal cracking above the tested wall strip was also observed, with cracking commencing at the wall top corners.

Following the completion of as-built wall testing, a CFRP NSM retrofit (repair) was applied, with the repaired wall tested using the same setup and boundary conditions as used for the as-built test. The pressure was increased until visible cracking within the vicinity of the strip occurred, and the airflow was then paused (without releasing the airbag pressures) so that crack locations could be observed and marked at different stages throughout the test. Numerous new cracks formed in the vicinity of the strip and the final crack pattern is illustrated in Figure 19. The pressure in the airbag was increased until a total mid-height displacement of 130 mm was reached. At this displacement substantial wide cracking of the masonry above the tested wall strip had occurred (shown in Figure 19 (c)) and due to safety considerations the pressure in the airbag was not further increased. The total lateral load – mid-height displacement response is shown in Figure 18.

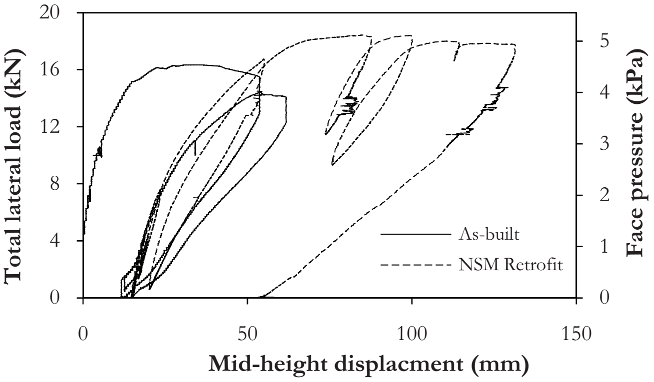
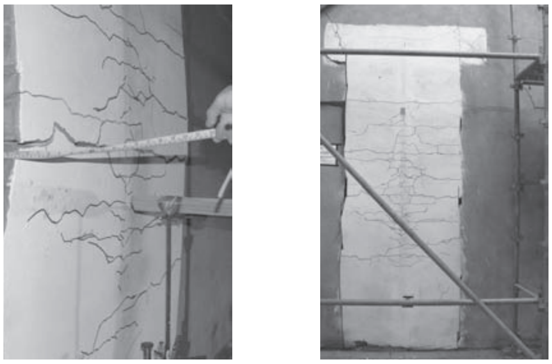


Figure 18. Total lateral force-displacement response for out-of-plane as-built and NSM repair tests



(a) Wall displacement and crack pattern

(b) Crack pattern above the test wall

(c) Crack pattern above the test wall

Figure 19. NSM repair – final crack pattern

7.4 Discussion

The location of crack formation at 53% of the strip height during the as-built test was comparable to the results obtained during laboratory testing of walls with similar geometry (Derakhshan et al. 2010). It is clear that the boundary conditions that are encountered in real buildings are different from those simulated in the laboratory. For the testing reported here the boundary condition at the top of the test wall restrained upward movement and resulted in arching action that caused cracks in the gable above the wall strip edges that propagated at approximately 45^{o} angles away from the wall strip corner. In the second semi-cycle, shown in Figure 18, due to the pre-existing cracks, the level of overburden had decreased and resulted in a 16% reduction in total lateral strength.

A single CFRP NSM strip was shown to increase the post-cracking wall strength, as illustrated in Figure 18, the wall strength increased by approximately 35%. Due to a large number of newly formed cracks it is clear that there was a larger amount of energy dissipation occurring compared to the as-built condition. Even though further research is required to accurately establish the design guidelines for this type of seismic retrofit, this and previous testing have illustrated that this retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components.

8.0 OUT-OF-PLANE SEISMIC ASSESSMENT

Multiple sources of uncertainty are involved with the evaluation of unreinforced masonry elements. These include arbitrary variation of material properties, of construction practice and of structural details. NZSEE (2006) proposes a methodology for the evaluation of URM walls loaded in a one-way bending condition. Although the out-of-plane tested wall in this research acted in a one-way bending condition, the behaviour was improved by the arching action arising from the top support configuration. Such arching action does not necessarily develop in an earthquake as any excitation involves the shaking of the building as a whole, and not an individual wall strip. A similar arching action also develops as a result of the contribution of the URM corners to the out-of-plane URM wall behaviour. With this analogy, the tested wall strip is assessed based upon the assumption that arching action that developed due to the existing masonry gable above the tested wall strip was comparable to that developed in walls supported by URM corners.

Based on NZSEE (2006) recommendations the subject wall was estimated to have an out-of-plane strength capacity of 39% of new building standard (%NBS). An overburden load equal to the weight of a masonry column above the tested wall was included in the calculations based on the NZSEE (2006) guidelines. The New Zealand Loading Standard (NZS 1170.5:2004) was used to calculate seismic demand, based on the assumption of a ductility factor of 1.0, an annual probability of exceedance of 1/500, a risk factor of 1, and a soil type of C. The seismic demand was then compared with measured performance of the tested wall, which demonstrated an out-of-plane strength of up to 129% of the seismic demand. Table 6 compares the results of this testing programme with that obtained using the NZSEE (2006) current recommendations, suggesting that the recommendations are overly conservative. The NZSEE (2006) procedure for out-of-plane assessment of one-way URM walls has been previously shown to be erroneous and conservative (Derakhshan et al., 2009).

Table 6. Out-of-plane assessment

%NBS (NZSEE 2006)	Strength demand, (kPa)	Capacity/Demand × 100	Max. wall face pressure, (kPa)
39	3.5	129%	4.5

9.0 CONCLUSIONS

The material properties of B-East building were determined by conducting in-situ testing and laboratory tests on extracted samples. The average brick compression strength was determined to be 19.4 MPa while the average three brick high prism compression strength was 9.6 MPa, with a Modulus of Elasticity of 2.45 GPa. It was concluded that mortar bed joint shear strength did not markedly increase with increasing level of axial precompression and that the cohesion values can be assumed as the average shear strengths at different levels of axial precompression, and are equal to 0.86 MPa and 0.19 MPa for the northern and southern gables respectively.

From in-plane diagonal shear tests on extracted wall panels from the B-East building it was shown that the laboratory-built wall panels composed of 1:2:9 mortar mix (cement:lime:sand) have maximum shear stress that closely matched the values obtained from the building samples. Excluding the pre-damaged wall panel AGW2, the average shear stress at cracking was 0.082 MPa and a maximum average shear stress of 0.092 MPa was obtained for B-East building wall panels.

Predictive equations for sliding failure provided by NZSEE guidelines and FEMA 356 overestimated the measured sliding strength of the wall section. The prediction by NZSEE guidelines provided a closer estimation of the actual value.

Out-of-plane in-situ testing of an as-built and a retrofitted URM wall using the NSM FRP technique was also conducted at the B-East building. The location of crack formation at 53% of the strip height was comparable to the results obtained during laboratory testing of walls with similar geometry. The out-of-plane wall behaviour subject to arching action was analogized with that of a wall subject to arching action developed by URM corners, and a comparison was made between the outcome of assessment using this experimental research and that using NZSEE (2006). The comparison suggested that the NZSEE (2006) guideline is overly conservative.

A single CFRP NSM strip was shown to increase the post-cracking wall strength by approximately 35%. Even though further research is required to accurately establish the design guidelines for this type of retrofit, this testing and further analysis have illustrated that this retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components.

The outcomes of such in-situ tests are of great assistance to understand the in-situ seismic response of heritage URM buildings, and the data acquired from such tests is essential to validate laboratory-based tests.

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A man owned a small farm in Australia.

The Fair Work Australia Office claimed he was not paying proper wages to his staff and sent a representative out to interview him.

“I need a list of your employees and how much you pay them,” demanded the rep.

“Well,” replied the farmer, “there's my farm hand who's been with me for 3 years. I pay him $500 a week plus free room and board. He also gets triple time for working on a Sunday and a slab of beer for a Happy Hour every Friday”.

“The cook has been here for 18 months, and I pay her $400 per week plus free room and board. She doesn't work on Sundays and I provide paid satellite television for free in her room”.

“Then there's the half-wit. He works about 18 hours every day and does about 90% of all the work around here. He makes about $20 per week, Pays his own room and board, and I buy him a bottle of whiskey every Saturday night. He also sleeps with my wife occasionally.”

'That's the guy I want to talk to...the half-wit,' says the agent.

'That would be me,' replied the farmer.