MONOTONIC TESTING OF UNREINFORCED AND FRP-RETROFITTED MASONRY WALLS PRONE TO SHEAR FAILURE IN AN EARTHQUAKE

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ABSTRACT:
A research programme has been initiated in New Zealand to address concerns regarding the seismic performance of New Zealand's unreinforced masonry (URM) building stock. As a component of this programme, monotonic in-plane shear (diagonal compression) tests were conducted on URM wallettes to simulate the commonly observed diagonal shear failure of in-plane walls in an earthquake. In total, four wallettes were tested, three of which were retrofitted with various configurations of GFRP strips. The unretrofitted wallette failed in a brittle manner. The retrofitted wallettes exhibited a more ductile behaviour. Significant increases in wallette strength and pseudo-ductility were achieved by the application of GFRP.

KEYWORDS:
GFRP, retrofit, seismic, shear, testing

1. INTRODUCTION

New Zealand is located at the edge of the Pacific and Australian tectonic plates. This unique location means that a large part of the country is subjected to frequent seismic activity. Many earthquakes have struck New Zealand in the last one hundred years, and resulting damage, though low when compared with other more populous countries, is significant for a nation of just 4.3 million people. The most destructive of these earthquakes was the 1931 Hawke's Bay Earthquake, which resulted in the deaths of 256 people and caused significant damage to the buildings in the affected area, most of which were unreinforced masonry (URM) buildings (Dowrick 1998). Due to the poor performance of URM buildings in that earthquake, the popularity of URM subsequently declined in New Zealand. Currently, URM construction is prohibited in New Zealand – the current masonry standard refers only to reinforced masonry elements – but a significant number of URM buildings remain. Most of these buildings were constructed in the period between 1880 and 1930. URM buildings are considered earthquake-prone and their presence in significant numbers in New Zealand is expected to result in undesired losses should a major earthquake strike New Zealand. In response to these concerns regarding the expected poor seismic performance of New Zealand URM buildings, a seismic retrofit research programme has been initiated in New Zealand. Research is focused on accurately assessing the seismic vulnerability of existing buildings and evaluation of retrofit solutions for seismically vulnerable URM buildings. The research in this paper was conducted as a component of the aforementioned research programme.

This research investigates the performance of GFRP material as a seismic retrofit solution for in-plane URM walls, which are likely to fail by the formation of diagonal shear cracks in an earthquake. In an earthquake, the in-plane URM walls elements may fail/deform in one of the following modes: (i) rocking; (ii) shear sliding; (iii) diagonal shear or (iv) compression (Magenes and Calvi 1997). The latter two modes of failure are brittle modes of failure and, if any preference for failure modes exists, are the ones to be avoided. The diagonal shear failure in a URM wall is generally exhibited by the formation of 'X' type cracks and takes place due to the exceedance of tensile strength of masonry. It is a commonly observed mode of failure in the walls and piers of earthquake-damaged buildings (Figure 1) (Javed et al. 2006). The 'X' shape of cracks arises due to the cyclic nature of the earthquake motion. The cracks may pass through either bricks or masonry joints or partly through both bricks and masonry joints. The crack path is determined by the relative strength of bricks, the brick-mortar interface and mortar. In the case of heavy damage to URM walls, the vertical load carrying capacity of the walls drops significantly.

Figure 1 Diagonal shear cracks in earthquake-affected URM walls and piers (Javed et al. 2006)

The test for determining the diagonal shear strength of URM has been standardised by the ASTM (ASTM 2002). The procedure involves rotating the URM wall by 45^{o} and applying the force vertically along one diagonal of the wall. Some researchers tested the URM specimens in accordance with the ASTM procedure (for example, Marshall et al. 2000). Others modified the standard method by keeping the wall vertical and rotating the loading jack to align it along the one wall diagonal. This latter approach has been adopted for this test programme, mainly due to the following two reasons:

1. In actual construction, load bearing walls are always vertical. The resultant force due to gravity loads and earthquake imposed forces may be in any direction.

2. A component of the gravity load acts along the masonry joints in case of walls that are not vertical. This is not the case in real construction and may lead to a drop in the wall shear strength.

A brief review of past research on this subject is given in the forthcoming paragraphs.

Li et al. (2005) performed diagonal shear tests on single-leaf unreinforced concrete masonry walls. The test walls were strengthened by FRP laminates, which were installed by either wet layup technique or using the near surface mounting (NSM) technology. The failure of walls retrofitted on only one face consisted of two phases: in-plane phase and out-of-plane phase. Most retrofitted walls failed by the formation of diagonal stepped cracks. Some walls, however, exhibited sliding shear failures by the formation of a horizontal crack along a masonry joint. The sliding shear failure occurred generally in the walls having FRP bars in alternate joints (i.e. not in all joints).

Turco et al. (2006) performed diagonal compression tests on six concrete masonry wallettes, each measuring 1600 mm × 1600 mm × 150 mm. One wallette was tested unreinforced, whereas the other wallettes were strengthened externally with GFRP bars in different arrangements. The unreinforced wall failed in a brittle manner. A strength increase of up to 123% was achieved with the application of GFRP bars. Maximum strength increase was obtained with the application of horizontal bars in every masonry joint. Pseudo-ductility was also substantially increased.

Research on the enhancement of the diagonal shear strength of URM by the use of FRP materials has shown that FRP is a suitable material for the enhancement of masonry shear strength and ductility. However, the previous work has focussed mainly on concrete masonry or single-leaf brick masonry, which is not representative of the construction in New Zealand. Load-bearing URM walls are at least two masonry leaves thick. Brick is the prevalent masonry unit in New Zealand. Also, only a few FRP retrofit schemes have been tested. In addition, retrofit schemes have been applied on walls having different masonry materials/properties. The assessment of the efficiency of FRP in the improvement of properties deemed essential for a better seismic behaviour becomes difficult in this situation.

2. RESEARCH PROGRAMME

In this research, diagonal shear tests were conducted on four URM wallettes, three of which were retrofitted with GFRP strips. Each wallette measured 1170 mm × 1170 mm × 225 mm and consisted of two masonry leaves. The wallettes were built in common bond pattern with header bricks every fourth course. Recycled bricks with nominal dimensions of 225 mm × 110 mm × 90 mm were used to build the test wallettes. The bricks were bonded together by means of a weak mortar, which consisted of 1 part cement, 2 parts lime and 9 parts sand, by volume. The masonry materials and bond pattern, and wallette thickness were selected to approximately replicate the conditions in old New Zealand URM buildings. The compressive strength of bricks, mortar and masonry prisms are given in Table 1. Tests for the determination of compressive strength of materials were conducted in accordance with ASTM standards C67-03a, C109-02 and C1314-03b.

One wall (DS-0) was tested as-built. Three walls (DSG-3, DSG-4, DSG-5) were retrofitted using different configuration of strips cut from glass fibre sheets (Figure 2, Table 2). The sheets were first saturated with epoxy and placed on the epoxy-cured wallette to form FRP. The details of the FRP application are given elsewhere (Mahmood et al. 2008). The composite had a tensile elastic modulus of 26.1 GPa and an ultimate tensile strength of 575 MPa (Fyfe Co. LLC).

Table 1 Compressive strength of materials

Material	*Mean compressive strength (MPa)*	*Coefficient of variation*
Brick	10.5	14.8
Mortar	3.6	44.7
Masonry	7.0	12.8

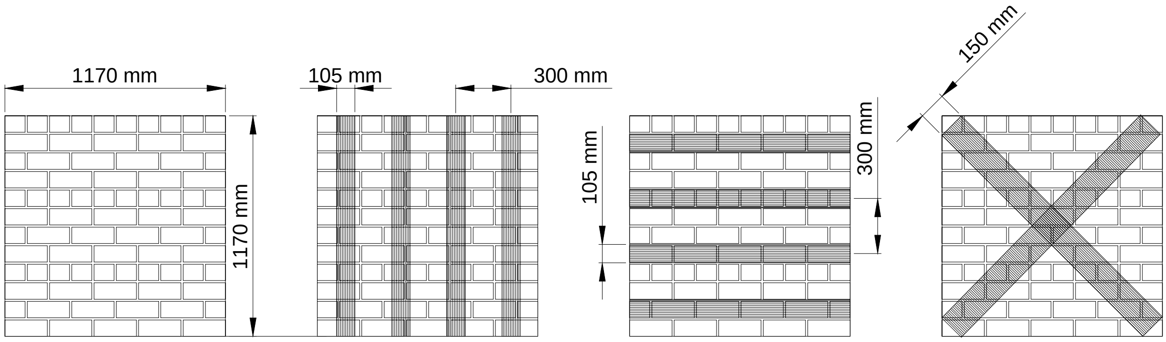


Figure 2 FRP configuration

Table 2 Strengthening schemes for test wallettes

Wallette	*Strengthening scheme*
DS-0	As-built (unretrofitted)
DSG-3	Four 105 mm wide vertical GFRP strips (two layers) at 300 mm on centres
DSG-4	Four 105 mm wide horizontal GFRP strips (two layers) at 300 mm on centres
DSG-5	Two 150 mm wide diagonal GFRP strips (two layers)

3. TEST SETUP

Figure 3 shows the test setup. The test wallettes were built separately and then moved to the test location. For testing, the wallettes were placed on a steel I-beam. The diagonal force was applied to a wallette by means of a hydraulic actuator through a steel shoe placed at the top corner and transferred to another loading shoe at the bottom corner through two high-strength steel bars placed along each wallette face. Diagonal displacements were measured using two perpendicular portal transducers aligned along the wallette diagonals. Force was applied in small increments until failure. Force and displacement measurements were recorded by means of a data acquisition system.

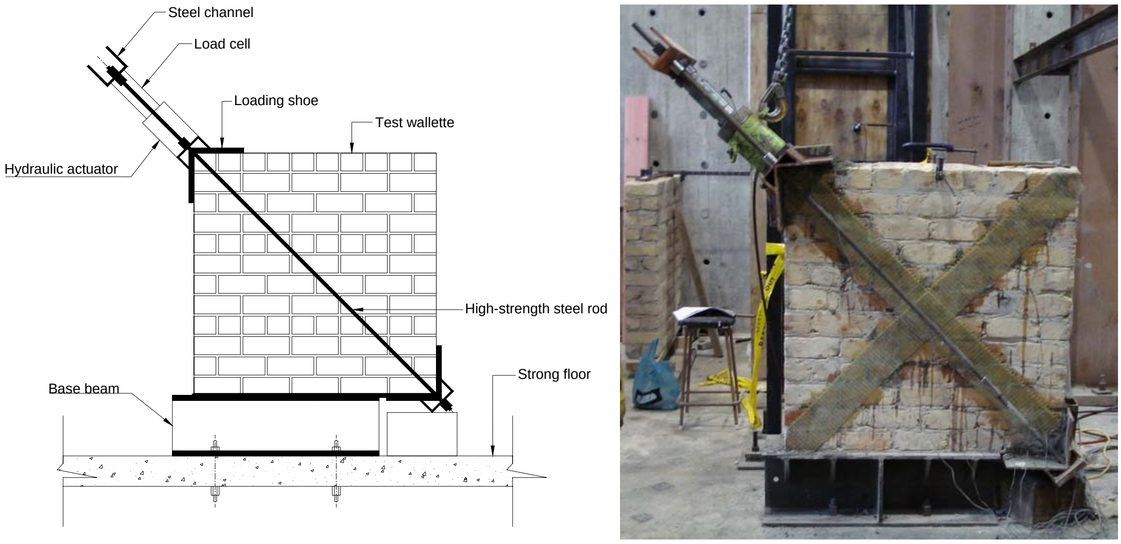


Figure 3 Test setup

4. EXPERIMENTAL RESULTS

In this section, a description of the behaviour of each test wallette under the imposed diagonal forces is given. Also, data regarding the peak diagonal and horizontal forces, shear strength, pseudo-ductility and ductility at 60% of the shear strength is provided. The horizontal component of the peak diagonal force, *H*, is calculated simply by multiplying the peak force, *F*, by the cosine of 45^{o}. Shear strain, γ, is determined as follows:

$$\gamma = \frac{\Delta_{short} + \Delta_{long}}{g} \tag{3.1}$$

where γ_{short} = diagonal shortening, parallel to the applied force; γ_{long} = diagonal elongation, normal to the applied force; and g = gauge length.

Shear strength is determined by dividing the peak horizontal force, *H*, by the effective area of a wallette, *A*. Effective area of a wallette, *A*, equals

$$A = tL \tag{3.2}$$

where t = wall thickness; L= wall length.

The shear strength versus shear strain relationship for the tested wallettes is shown in Figure 4.

Pseudo-ductility, μ, has been previously defined as the ratio of the shear strain at ultimate and the shear strain corresponding to the bend-point of the shear stress versus shear strain graph (Li et al. 2005). This value, μ, has been determined here for each wallette. Since some tests were stopped before failure (i.e. the drop of the diagonal force to zero) to prevent damage to the equipment, another concept of pseudo-ductility corresponding to 60% of the diagonal force is introduced here. Pseudo-ductility at 60% of the peak force, μ_{60}, is defined as the ratio of the shear strain corresponding to 60% of the peak force to the shear strain at the bend-point of the shear-stress versus shear strain curve. Shear modulus, *G*, for each wallette is determined from its shear stress – shear strain curve as the chord modulus between 0.05 and 0.33 of the shear strength. These values are tabulated in Table 3 for all specimens.

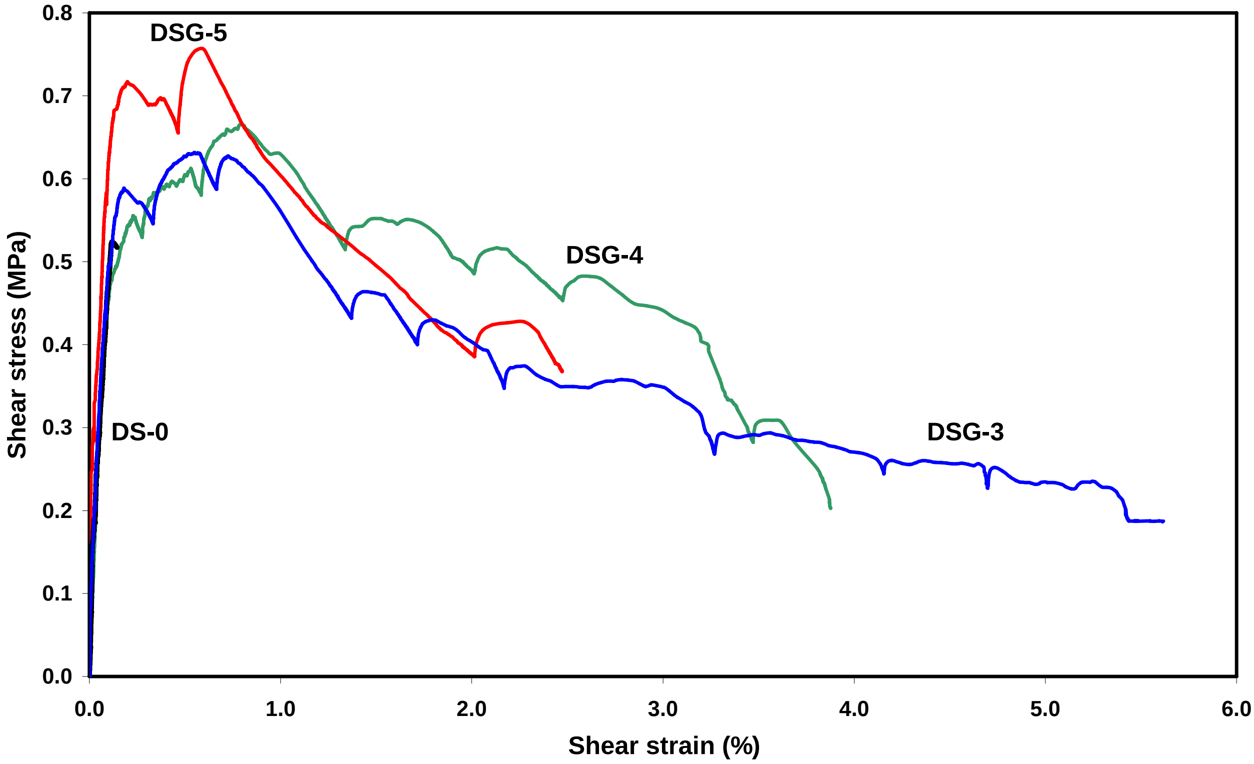


Figure 4 Shear stress versus shear strain curves for the test wallettes

Table 3 Test results

Wallette	*Peak diagonal force, P (kN)*	*Peak horizontal force, H (kN)*	*Shear strength (MPa)*	*Shear modulus, G (GPa)*	*Pseudo-ductility, μ*	*Pseudo-ductility corresponding to 60% of the peak force, μ_{60}*
DS-0	194	137	0.52	0.62	1	1
DSG-3	234	165	0.63	0.67	29.6	11.2
DSG-4	243	172	0.67	0.54	25.9	21.6
DSG-5	280	198	0.76	1.67	11.8	7.9

Wallette DS-0: Wallette DS-0, was the control wallette. It failed by the formation of a major diagonal shear crack (Figure 5a). The crack passed mainly through the head and bed masonry joints, however, some bricks also cracked. The failure occurred in a brittle manner immediately after the formation of the diagonal crack. The peak diagonal force was 194 kN. The corresponding compressive diagonal displacement was 1.41 mm (0.12% shear strain). This wall had a pseudo-ductility, μ, and pseudo-ductility at 60% of the peak load, μ_{60}, equal to 1.

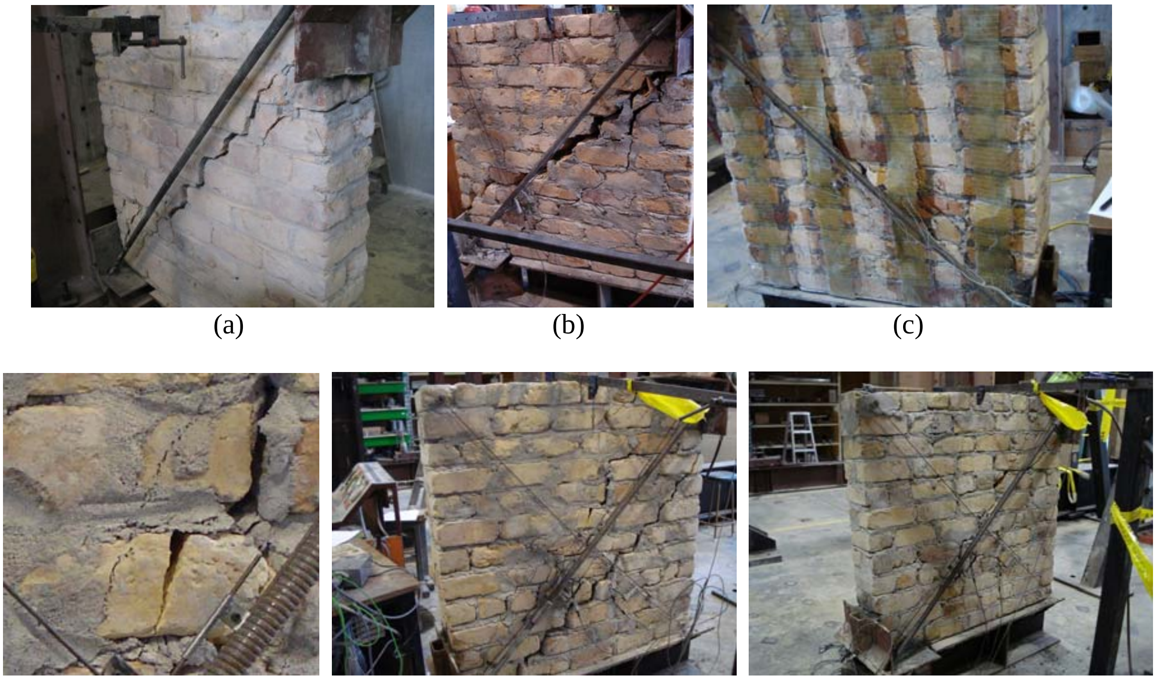
Wallette DS-3: Wallette DS-3 was retrofitted on one face only by four vertical GFRP strips. The failure of this wallette initiated by the formation of a diagonal crack in the direction of the applied force (Figure 5b). The crack developed at a displacement of 2.20 mm (0.18% shear strain) between the vertical strip closest to the point of application of force and the next strip. A second crack parallel to the first crack, but not running across the full length of the wallette, appeared at a compressive displacement of 5 mm (0.79% shear strain). Debonding of the second strip (with respect to the point of application of force) started at a diagonal displacement of 10.80 mm (2.16% shear strain) at the mid-height of the strip. With increasing displacements, the other strips also started to debond (Figure 5c).

Horizontal rupture of some strips also occurred near the location of the main crack. At 34 mm compressive displacement, the portal transducer reached its capacity and was removed. The test was stopped immediately afterwards to protect the equipment as there was significant out-of-plane wall displacement, which increased the hazard of the overturning of the wallette. The out-of-plane displacement phase followed the initial in-plane displacement phase. The peak diagonal force was 234 kN at a compressive diagonal displacement of 3.94 mm (0.57% shear strain).

Wallette DS-4: This wallette was retrofitted with on one face only by four horizontal GFRP strips. Failure of this wall also took place due to the occurrence of a major diagonal shear crack. The crack passed mainly through the masonry head joints, but a few bricks were also damaged (Figures 5d and 5e). This crack appeared at a diagonal displacement of 3 mm (0.47% shear strain). A second crack parallel to the first crack appeared at a displacement of 5 mm (1.05% shear strain). Debonding of the bottom-most FRP strip took place after the formation of this second crack. Testing was stopped after the development of large out-of-plane displacements. The peak diagonal force was 243 kN at a corresponding displacement of 4.08 mm (0.79% shear strain).

Wallette DS-5: Two GFRP strips were applied along the wall diagonals on one face only to retrofit this wallette. Upon loading, a diagonal crack formed under the diagonal strip parallel to the applied force at a displacement of 2.3 mm (0.20% shear strain) (Figure 5f). At a higher value of displacement, another crack appeared parallel to the first crack but outside the GFRP strips. This second crack did not extend the full length of the wall and joined the first crack roughly at the centre of the wall. The force dropped quickly after the formation of the second crack. The test was stopped immediately afterwards due to large out-of-plane displacement and rapid deterioration of the wallette. The wallette reached a peak force of 280 kN at a displacement of 3.70 mm (0.60% shear strain).

Except for the as-built wall DS-0, which failed in a brittle manner, all other (retrofitted) walls failed gradually by the formation of more than one diagonal crack. The initial crack occurred just before the wall reached the peak force. The force dropped at a higher rate after the formation of the second crack. All retrofitted walls reached their strength at a shear strain between 0.57% and 0.79%. The retrofitted walls also displayed large out-of-plane displacements.



(a) (b) (c)

(d) (e) (f)

Figure 5 Test wallette failures: (a) DS-0; (b) DSG-3; (c) DSG-3 FRP debonding; (d) DSG-4, crack passing through brick and masonry joint; (e) DGS-4; and (f) DSG-5.

5. CONCLUSIONS

FRP is a viable option for the seismic retrofit of URM structures. Significant increases in strength and pseudo-ductility were achieved with the application of GFRP strips in different configurations. Maximum strength increase was achieved with a diagonal configuration of GFRP strips. This strength increase amounted to 46%. This specimen was also the stiffest of all specimens. However, the failure of this wallette was quite brittle, with the pseudo-ductility equalling just 11.7. The increase in shear strength was less remarkable with other GFRP arrangements, but failure was more ductile in each case. The most ductile failure was obtained with vertical GFRP strips; the pseudo-ductility was equal to 29.6. The gradual debonding of GFRP strips in this wallette helped to achieve a ductile failure. This is useful in providing an early failure warning to the occupants of an earthquake-prone building during an earthquake. The post-peak rate of drop of force up to 60% of the peak force was the lowest for the wallette DSG-4, as is evident by its largest μ_{60} value. Large out-of-plane displacements were observed in all retrofitted walls. This was due to the application of FRP retrofits on only one wall face. This is the only practical retrofit solution for most buildings of heritage importance. Large out-of-plane displacements can be useful in providing the occupants of a building with early signs of building failure.

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